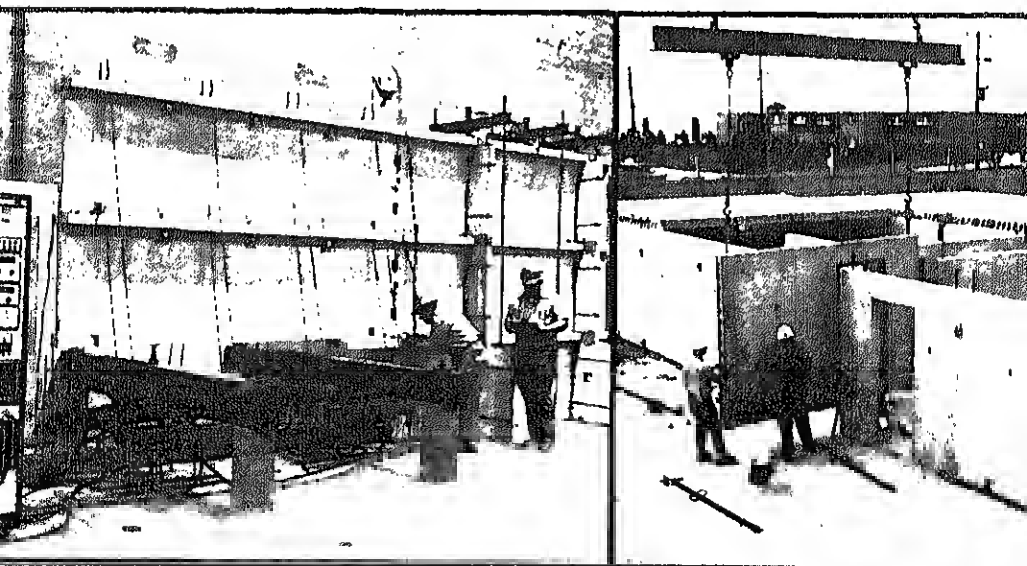




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# Philosophy of Structural Response Normal and Abnormal Loading

Aug.



# DESIGN AND CONSTRUCTION LARGE-PANEL CONCRETE STRUCTURES

Contract

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## Philosophy of Structural Response Normal and Abnormal

Author

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Structures which can affect their behavior are identified. Methods reducing the risk of progressive collapse from abnormal loads are examined. A philosophy of design based on bridging local damage is produced which emphasizes continuity and ductility within and across connections. A methodology is developed to be used in establishing minimum detailing criteria to ensure integrity of the structure. In this General Structural Integrity approach code requirements for minimums will be provided to assure an alternate path for the load in the event of a local failure of a primary load bearing element. Design considerations for both individual elements and connections are discussed conceptually.

purpose of this report is to discuss qualitatively the response of the structure to both normal and abnormal loading conditions. Specifically, the report:

1. Discusses conceptually, conditions inherent in LP structures which can effect the procedures used in the analysis and design for normal loading conditions.
2. Develops a methodology to be used in establishing minimum detailing criteria (indirect design) to ensure structural integrity of the building thus accommodating the effects abnormal loading conditions.
3. Discusses details in connections and elements which can affect the behavior of the structure.

The design of each element under the applicable gravity and lateral loadings should satisfy the specified conditions of "ultimate strength and serviceability. For traditional or normal loadings the stresses accounted for in design are principally limited to flexure and shear in the horizontal elements and compression, flexure and shear in the vertical elements. In general, methods and concepts used in the design of conventional structures to resist normal loadings are applicable to LP structures. However, to ensure satisfactory performance, the engineer must pay particular attention to the effect of connection details on the overall behavior.

In an LP structure, unless special details are employed to ensure continuity at the supports, the vertical loads are carried by the frame in a simple post and beam fashion. In the design of floor elements the engineer should consider serviceability requirements including:

all assemblies, the simplified stiffness method is used when the walls are solid and vertically continuous; if the walls are coupled over openings, the shear connection, frame analysis or finite element method is used. Under lateral loads, serviceability conditions which should be considered include lateral deflection and dynamic motion as it affects occupant comfort.

When a local failure due to an abnormal loading is not confined to the area of initial distress, but spreads either horizontally or vertically through the structure, it is termed a progressive collapse. For abnormal loads it is not the loading but the susceptibility of the building to progressive collapse which presents the real risk. Two modes of progressive collapse are defined: that caused by the inability of the structure to form an alternate path to bridge local damage, and that caused by insurmountable debris loading.

Three possible approaches can be employed to reduce the risk of progressive collapse: eliminate the hazards which cause local failure; design the structure so that the hazard does not cause any local failure; or allow the local failure to occur but ensure an alternate path for the load. Since the nature and magnitude of most abnormal loads are unpredictable, the third approach is advocated. When a structure has the ability to bridge over local failure it has what is termed General Structural Integrity, the principal elements of which are continuity and ductility of members, of connections, and of the structure as a whole. This additional continuity and ductility is not usually required to resist normal loads, but allows the structure to mobilize the reserve strength needed to resist progressive collapse.

imum detailing is predicated upon:

1. Extent of damage -- an assessment of the nature and extent of local damage that is likely to occur under abnormal loads;
2. Alternate paths -- evaluation of alternate structural action which can develop as a consequence of a local failure to establish load flow in the remaining undamaged structure; and stability analysis of the partially damaged structure.

Based on this procedure a rationale for horizontal and vertical ties is established:

1. Transverse ties to develop cantilever and beam action in panels;
2. Longitudinal ties to develop partial membrane action in elements;
3. Vertical ties to develop suspension action in wall panels to ensure adequate shear capacity in the horizontal connections between panels; and
4. Peripheral ties to develop diaphragm action.

The combination of system continuity and ductility should enable the structure to absorb the abnormal load with minimal damage, or alternatively to bridge localized damage as a result of the abnormal load. Thus provision of General Structural Integrity eliminates the need

any part of every building.

In LP structures, as in any precast building, allowance must be made in the design for dimensional deviations. To reduce the need for onsite modification and adjustment the engineer should adopt and specify a "tolerance" system. A tolerance system is suggested in the report, based on a survey of several systems used in North America.

In the design of wall panels the engineer should consider the following: load and geometric eccentricities, slenderness, thermal effects, and effects of connections and openings. Similarly in floor elements, the effects of wall clamping on simply supported plank, horizontal shear composite systems and end bearing are among the more critical factors which should be examined.

By its very nature, an LP building is only as strong as the connections between the individual elements. If sufficient strength and ductility is not developed in the connection, the available strength in the adjoining elements may not be fully utilized. Within the report, strength and performance requirements for the most widely used connection details are discussed to provide a basis for analysis and design.

Connections are classified with respect to location, direction, and function. The primary connections discussed are: interior horizontal wall-to-floor; exterior horizontal wall-to-floor; horizontal floor-to-floor; and vertical wall-to-wall. Variables affecting connection behavior include: precast and cast-in-place concrete properties, floor penetration, load eccentricities, floor moment and mortar packing. The function and influence of each connection on the overall behavior of the structure is discussed.

conventional cast-in-place structural wall (egg crate) construction. Large panel buildings are differentiated by the general arrangement (Fig. 2) of load-bearing walls:

- (a) Cross wall system: in this most prevalent form, the load-bearing cross walls are perpendicular to the longitudinal axis of the building.
- (b) Spine wall system: the load-bearing walls are parallel to the longitudinal axis of the structure.
- (c) Mixed systems: a combination of cross wall and spine wall systems.

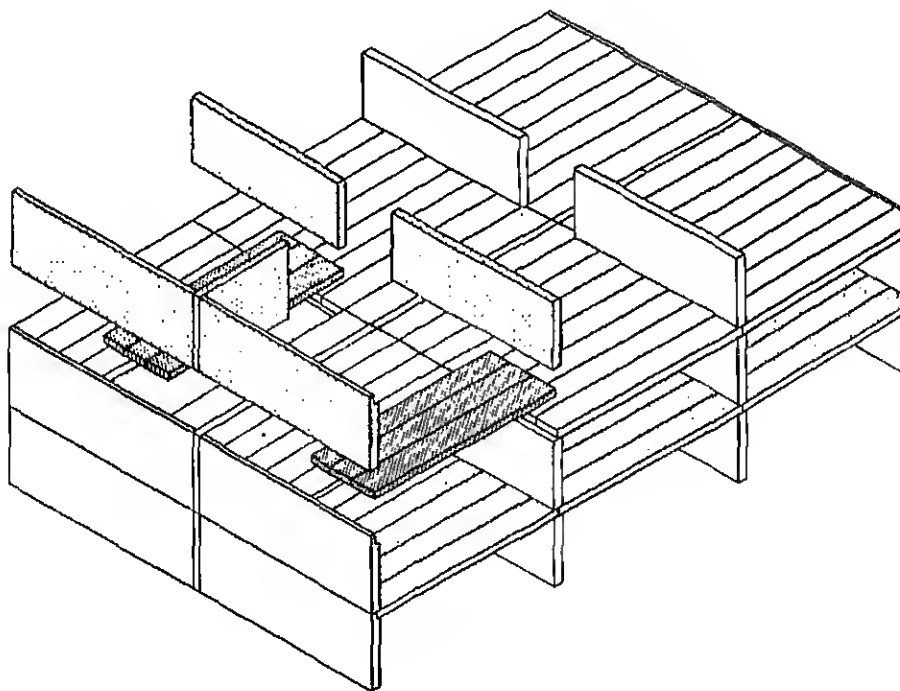


Fig. 1 Isometric View of Typical Large Panel Structure



(a) Cross Wall System



(b) Spine Wall System

Fig. 2 Typical Arrangement of Structural Wall Panels in Large Structures

In most LP systems, the walls transfer their loads directly to the substructure without an intermediate frame. This form of construction restricts open plans at any level and is thus most typically suited for multistory housing where walls of substance usually have to be provided between apartments to resist fire and noise transmission. Construction types considered under this investigative program include solid, sandwich, ribbed, hollow core or composite wall panels; and solid, hollow core, or ribbed floor units with or without cast-in-place topping. All elements can be either prestressed or conventionally reinforced.

The overall program objective is to develop minimum criteria for the design and construction of large panel structures. These criteria are being developed to ensure the structural safety and serviceability of residential buildings, while also providing minimum performance requirements to be used by designers and developers of innovative systems. The development of the criteria will also expand the knowledge of design and construction of large panel structures to a level comparable with that existing for conventional cast-in-place concrete or steel structural systems.

...sessed by ultimate strength, and serviceability as expressed in deflections, cracking and vibrations, should be considered not only for the structure as a whole, but also for its individual components, joints and materials.

As discussed in Report 1<sup>(1)</sup> all loadings may be classified under two categories:

1. Normal loading: a loading condition included in the typical analysis and design of a building, following current codes and practices; and
2. Abnormal loading: a loading condition (usually of high localized intensity) which is not explicitly included within current codes and practices.

of the loads to which a building may be subjected during its life span is probabilistic in nature.<sup>(1-5)</sup> Consequently, it would be desirable to perform theoretical safety analyses on the basis of probabilistic methods.<sup>(6-9)</sup> Currently much work is being carried out on probabilistic methods.<sup>(10-13)</sup> However, practical application of probabilistic methods to structural design does not seem imminent. The practicing engineer is not yet expected to use such methods in daily analyses and designs; rather, the conventional deterministic and semi-probabilistic methods introduced in today's codes and standards are used.<sup>(14-20)</sup>

to the introduction of probabilistic methods which would replace deterministic values, it is essential that all possible loading conditions be considered to some degree.<sup>(6,8)</sup> In the case of codes and standards for conventional steel and cast-in-place concrete buildings,



basis for evaluation (abnormal loadings) are handled indirectly through minimum detailing criteria for continuity and ductility of individual elements and their connections, and through the choice of safety factors for strength. Thus, indirect code provisions attempt to establish an adequate response of the structure to abnormal loadings. This indirect approach in traditional structures usually yielded an inherent ability to cope with abnormal loading conditions. In effect, a minimum level of integrity was established in most of our conventional structures.

This report is the second in a series of reports leading to the development of a "Recommended Practice for the Design and Construction of Structures." The purpose of this report is to discuss qualitatively the response of the structure to both normal and abnormal loading conditions. Specifically this report will:

1. Discuss conceptually, conditions particular to LP structures which can affect the procedures used in the analysis and design for normal loading conditions.
2. Develop a methodology to be used in establishing minimum detailing criteria (indirect design) to ensure structural integrity of the building thus accommodating the effects of abnormal loading conditions.
3. Discuss details in connections and elements which can affect the behavior of the structure.

- live load,
- soil and hydrostatic pressures,
- snow load,
- earthquake load, and
- wind load (excluding tornado).

Most building codes require consideration of volume changes due to temperature, shrinkage and creep. However, these are not routinely considered by the engineer, due to a lack of commonly accepted design procedures.

As noted in Report 1, the characteristics and magnitude of codified normal loading conditions are directly applicable to LP construction. A discussion of response of LP structures to the above loadings, and appropriate methods of analysis and design follow. Excluded from discussion are "earthquake load" and "effects of volume changes", which will be considered separately within subsequent reports.

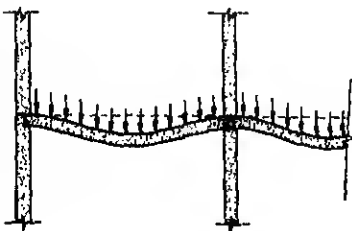
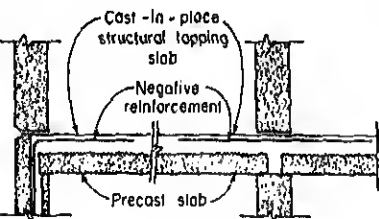
LP buildings are bearing-wall structures. Such structures, consisting of interconnected walls and slabs, are laterally stiff and can be forced without difficulty to resist normal loading conditions. An assessment of the nature of the traditional loads to be discussed in this section indicates two distinct categories of loadings: vertical (gravity loads) and horizontal or lateral loads. The design of each element under the applicable gravity and lateral loadings should satisfy the specified conditions of "ultimate strength" and serviceability.

Under vertical and lateral loads the stresses to be accounted for in design within the individual elements are principally limited to flexure and shear in the horizontal elements, and compression, flexure and shear in the vertical elements. Design methods to ensure adequate stress

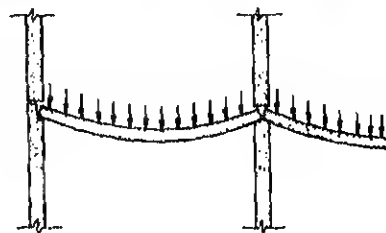
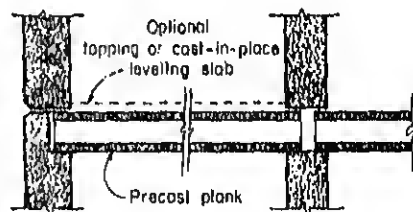
ction with an elastic analysis to determine design forces and defor  
tions.

### 2.2.1 Resistance to Vertical Loads

In conventional cast-in-place concrete slab systems it is customary to utilize flexural continuity at the supports in resisting gravity loads. However, an LP building subjected to such loads cannot develop flexural continuity at the supports unless a cast-in-place topping slab is employed, or reinforcing steel loops protrude from the slabs and walls to interlace within joints. In the case of topping slabs, sufficient thickness to accommodate the reinforcing and provide adequate bond to the precast floor elements is required to develop negative moment continuity (Fig. 3a). Where the structural cast-in-place topping is omitted and no interlacing loops are used, gravity loads are carried in a simple post-and-beam fashion, the slabs being simply supported (Fig. 3b).



Continuous Composite



(b) Simple Span Non-Composite

In the prestressed plank typically used in LP construction, crack control is not usually a problem; however, deflection can be a critical consideration. Computations must take into account the deflections due to both immediate and long-term effects. Slab vibrations at relatively high span-to-depth ratios should also be investigated. The short-term, immediate deflections should be calculated using the moment of inertia of the gross (uncracked) concrete section. The additional long-term shrinkage, creep and steel relaxation deflections of prestressed concrete members should be computed for the stress level due to sustained loads. The total allowable deflection should not exceed the limits stipulated in ACI 318-71.<sup>(15)</sup>

### 2.2.2 Analysis Techniques for Vertical Loads

Current criteria, formulae, and specifications contained within codes<sup>(14-20)</sup> and related literature<sup>(21-27)</sup> for the investigation of strength and serviceability are based on established analytical techniques. Consequently, a general discussion of gravity load resistance of LP buildings is not included here. However, specific design details pertaining to LP construction which may affect the strength and serviceability of the completed building will be discussed in detail within subsequent sections of this report.

### 2.2.3 Resistance to Lateral Loads

Continuous vertical wall elements in both the transverse and longitudinal direction of LP buildings are the basic structural elements to resist lateral loads. Although frame action is feasible and sometimes used in one of the orthogonal directions of the

related to the center of rigidity of the entire structure must be considered when distributing loads to these elements. Where the center of rigidity of the resisting elements and the center of mass of the structure do not coincide, torsion will be introduced into the structure. Because torsion due to lateral loads can produce significant stresses, particularly in peripheral elements, a more or less symmetrical arrangement of the principal lateral load resisting elements should always be attempted. If torsion due to asymmetry is unavoidable, then the analysis and design should reflect this condition.

In addition to strength requirements, the following serviceability conditions should be considered:

- (a) lateral deflection of the structure, particularly as this may affect the stability and cracking of structural and nonstructural elements; and
- (b) motion of the structure as it may affect the comfort of occupants.

Lateral deflection or drift is the magnitude of displacement at the top of a building relative to its base. The ratio of the lateral deflection to the building height is referred to as the "deflection index". Maximum allowable drift is imposed to limit possible adverse effects on the stability of individual wall elements as well as the structure as a whole, and to preserve the integrity of nonstructural partitions, glazing, and mechanical elements in the building. No systematic study has yet been published to determine the precise relationship between drift and the above factors.

To date only the Uniform Building Code<sup>(15)</sup> and the National Building Code of Canada<sup>(20)</sup> among North American model building codes

or exceed this criterion appears to have been satisfactory with respect to the stability of the individual elements and the structure as a whole. Recent studies have indicated that for tall concrete buildings containing structural walls\*, computed deflection indices ranging from 1/800 to 1/1200 are common and easily attainable.<sup>(30,31)</sup> For LP buildings constructed with structural wall elements in both directions, it would appear reasonable to expect a deflection index of at least 1/1000.

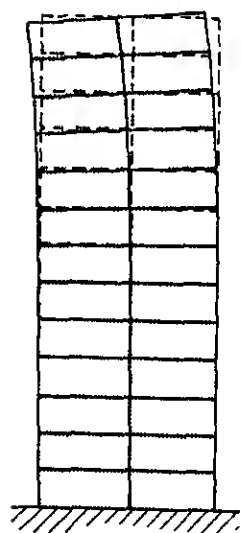
The sway motion of a tall building under turbulent wind, if perceptible, can produce psychological effects which render the building undesirable from the user's viewpoint. The sensation of motion which can be disturbing to an occupant of a building may result either from the visual perception of a relative displacement, or from the acceleration of the floor on which the observer stands. It has been noted<sup>(30,31)</sup> that no perceptible motion has been reported in residential, shear wall concrete buildings to date. This is apparently attributable to the high rigidity (an associated low deflection index--1/1000) inherent in tall structural wall buildings. Again, considering that LP buildings are quite stiff, there is little likelihood of detrimental effects from sway.

#### 2.2.4 Analysis Techniques for Lateral Loads

In an overall analysis of the lateral resistance of an LP structure, certain assumptions must be made about the effect of connections. Wall assemblies in LP buildings, consisting of story-high wall elements and horizontal and vertical connections, are non-homogeneous in nature. Therefore, to determine the strength and stiffness of a wall assembly, the effect of joint characteristics

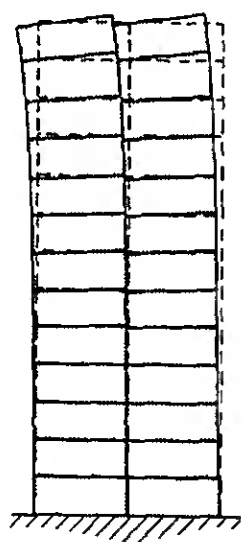
working stress levels. In calculating the increased shear resistance of the horizontal connections due to vertical load, allowance should be made for the effects of sustained gravity loads only.

A wall unit may be a solid wall composed of single or multiple panels or a series of separate walls connected either nonrigidly or rigidly by beams. In nonrigid linkage, the connection beams are usually within the thickness of the floor construction with openings equal to a story height. Rigid linkage is usually created by portions of the wall, thus openings are less than story height. Because of high bending and shear stresses, careful attention is usually given to the design of such connecting beams.



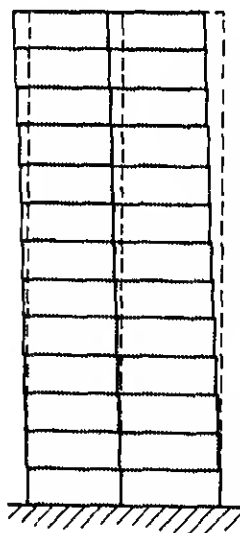
(a)

Monolithic



(b)

Non-rigid Vertical  
Connections



(c)

Non-rigid Horizontal  
Connections

openings involve simple bending theory only. Similarly, in structures whose walls are nonrigidly linked together with lintels incapable of moment transfer (typically at corridors), the lateral loads are distributed on the basis of relative stiffness (Fig. 6)

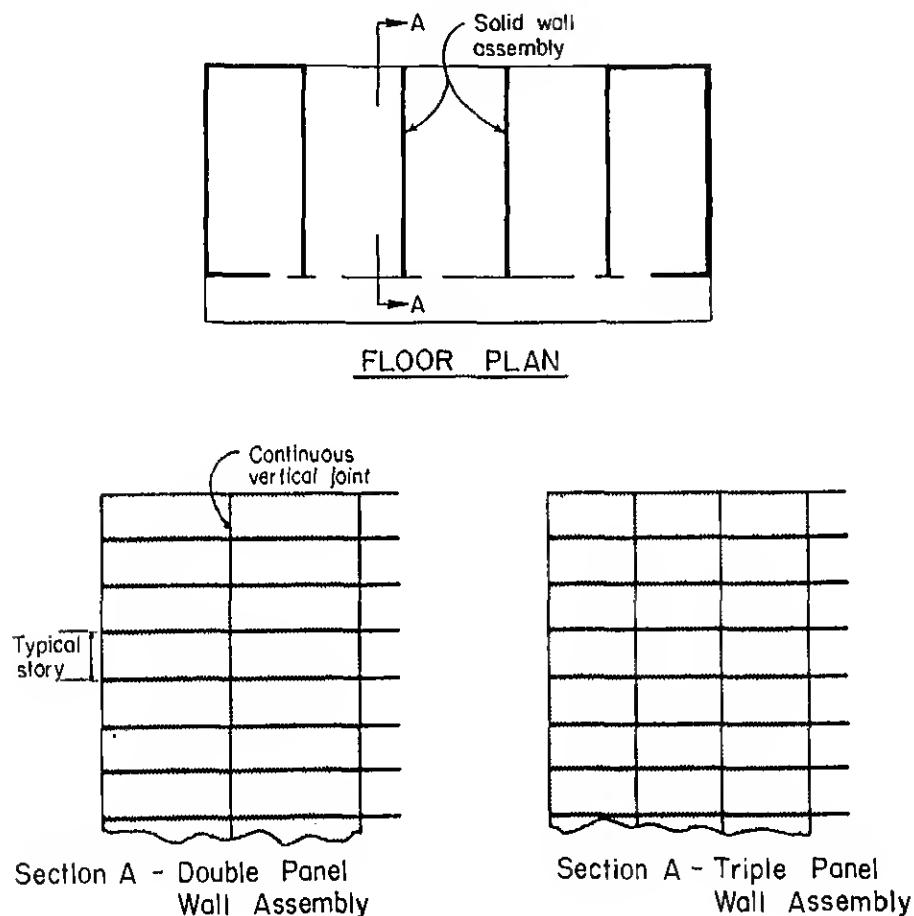


Fig. 5 Solid Wall Assembly



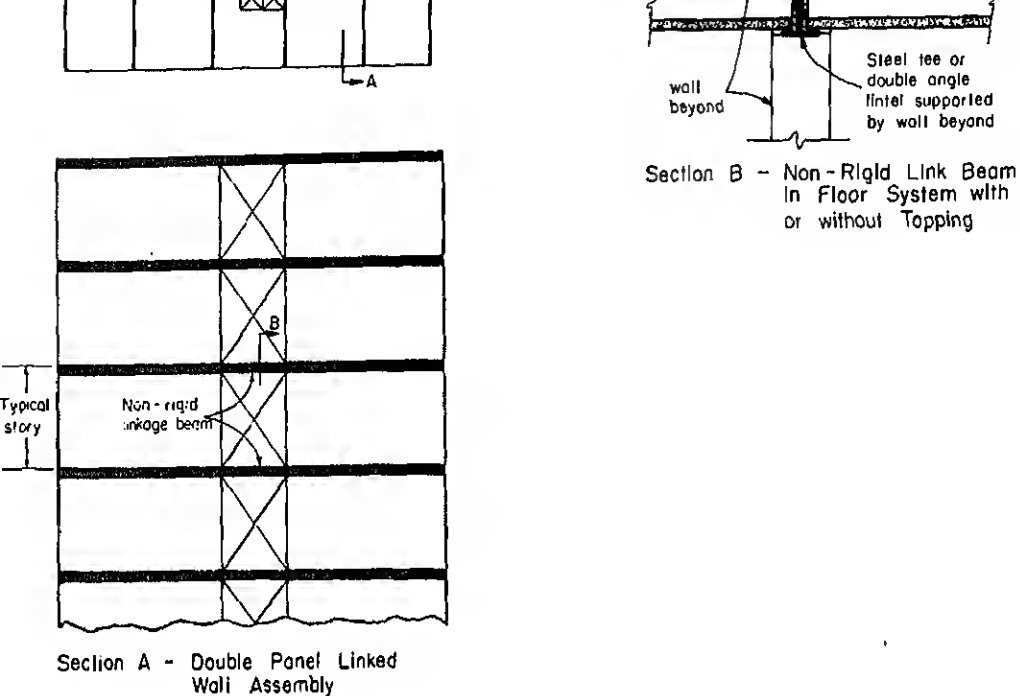


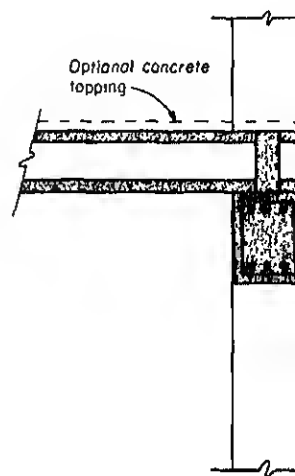
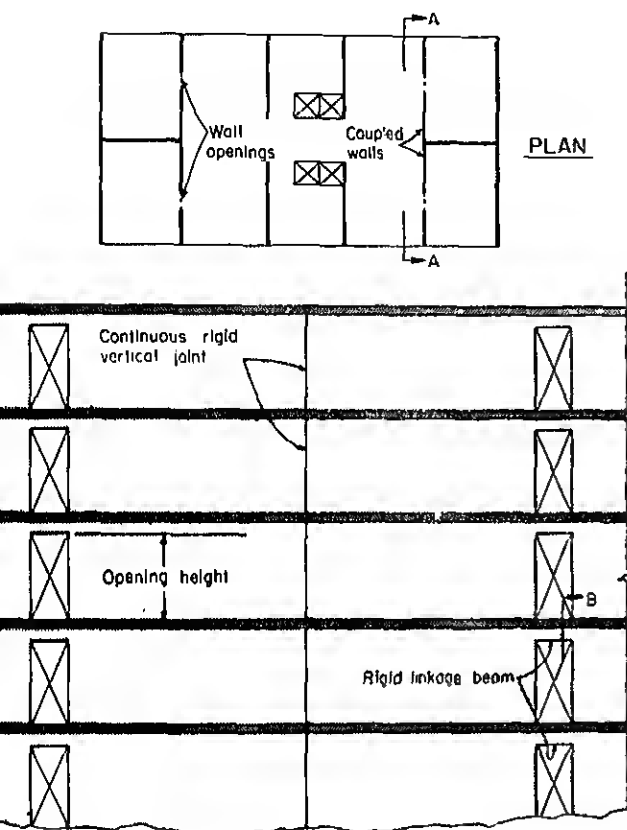
Fig. 6 Linked Wall Assembly

The distribution of lateral load for the systems described are based on the assumption that floors remain rigid within their planes, so that each wall assumes the same deflected shape. If a uniformly distributed lateral load, "W", is applied, the loading is distributed in the proportion:

$$\frac{I_1}{\Sigma I} W : \frac{I_2}{\Sigma I} W : \frac{I_3}{\Sigma I} W : \dots$$

to each wall.  $I_i$  is the moment of inertia of the wall,  $i$ , and  $\Sigma I = I_1 + I_2 + I_3 + \dots$  (shear deformation is neglected).

analysis must take this factor into account. (52-54)



Section A - Double Panel Coupled Wall Assembly

Fig. 7 Coupled Wall Assembly

The proportion of the total lateral load which each coupled wall assembly resists can be determined by inserting the deflection reciprocals in place of the moments of inertia in Eq. 2.1:

$$\frac{1/\delta_1}{\delta_1/\delta} W : \frac{1/\delta_2}{\delta_2/\delta} W : \frac{1/\delta_3}{\delta_3/\delta} W : \dots$$

Openings in wall units generally occur in vertical rows (Fig. Typically, the connection with the wall panel is provided by a reinforced beam which forms part of the wall unit itself (Fig. Sec. B). The terms "coupled shear walls", "pierced shear wall" and "shear wall with openings" are used to describe such wall assemblies.

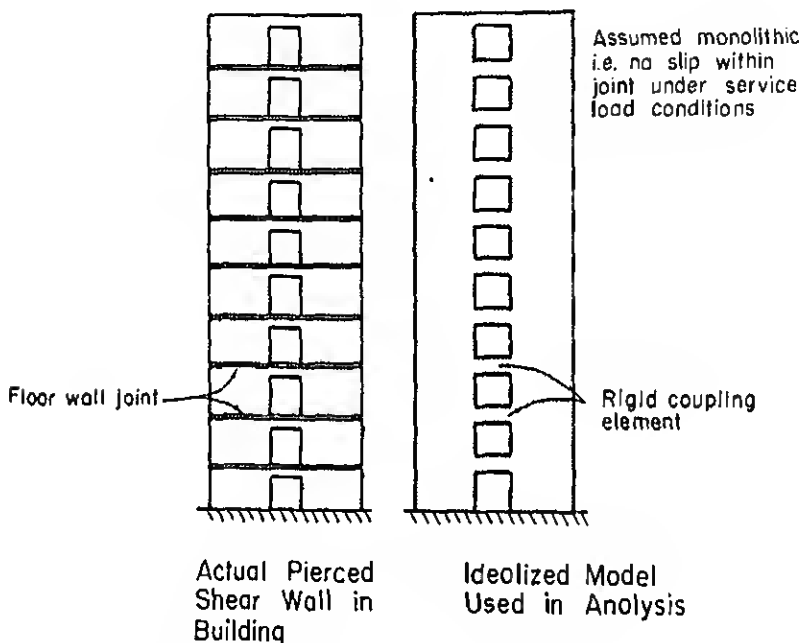


Fig. 8 Idealized Coupled Wall Assembly

If the openings are small (such as windows), their effect in a structural wall may be minor. Larger openings have a more pronounced effect and can result in a system in which frame action predominates. The degree of coupling between two walls has been expressed in terms of a geometric parameter,  $\alpha$ , having a unit  $1/\text{length}$ , which gives a measure of the relative stiffness of the connecting beams with respect to that of the walls. The parameter

separate cantilevers. For all intermediate values of  $\alpha H$ , the stiffness of the connecting beam should be considered.

Prior to the early 1960's, no practical analytical techniques for coupled walls were available. Consequently, coupled walls were designed as individual cantilevers, each carrying lateral load in proportion to its stiffness (moment of inertia). Although these structural walls were substantially overdesigned, the approach resulted in underdesigned connecting elements. Since the 1960's, considerable amount of information has become available<sup>(35-43)</sup> and a number of practical approaches have been suggested. Reference presents a detailed comparison of the various analytical approach

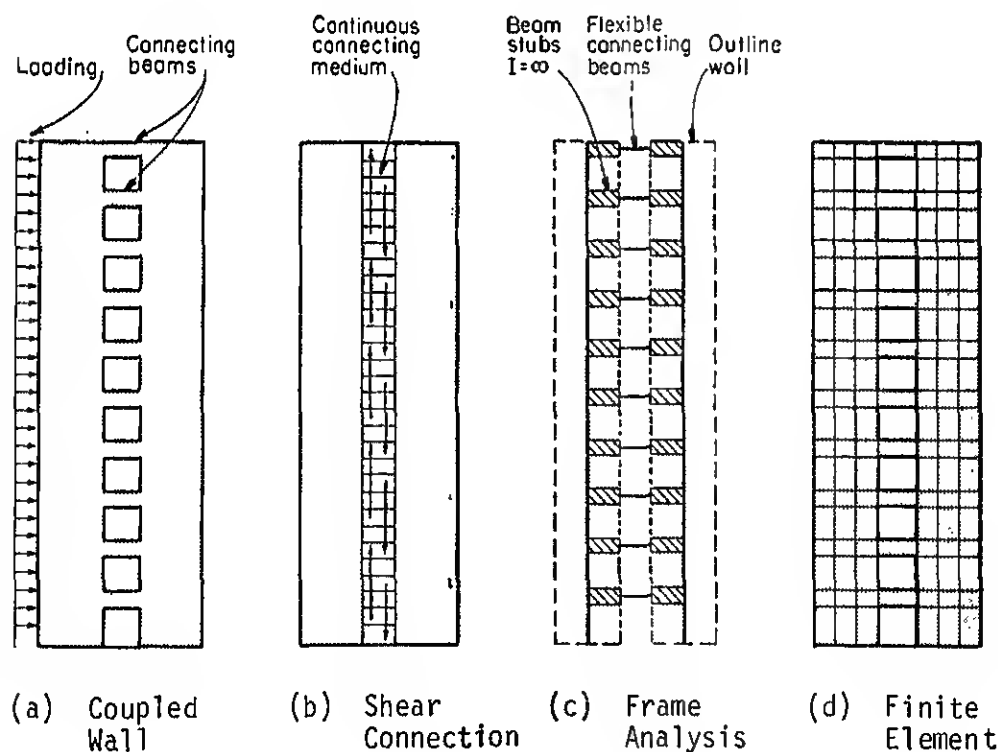


Fig. 9 Analytical Techniques for Coupled Walls

is based on a second order differential equation which gives deflection, moment and shear.

Possible sources of inaccuracies in the method are: local wall deformation,<sup>(44)</sup> greatly differing wall stiffnesses,<sup>(45)</sup> and reduction in stiffness of coupling elements due to cracking.<sup>(46)</sup> While the method has certain limitations, the shear connection method can be used to investigate the basic behavior of coupled wall systems.

Frame Analysis Method--Analytical methods employing computers are more accurate and considerably more flexible as they can take into account many more variables than the shear connection method. A computer technique analyzes a coupled wall as a frame, recognizing the finite width of the columns in comparison with the beams.<sup>(47)</sup> Infinitely stiff beam stubs from the centerline of the wall to the edges of the actual opening are introduced as illustrated in Fig. 9c. This method takes into account changes in wall thickness, story height, and concrete strength within the height of the building.

Finite Element Method--The technique known as finite element analysis considers the wall to be divided into a mesh of two-dimensional elements (Fig. 9d). By imposing the appropriate boundary conditions a solution can be obtained by matrix techniques which involve the solution of many simultaneous equations. The accuracy depends on the fineness of the mesh and the sophistication of the finite element, each of which in turn affects the computer running time.

### 2.2.5 Tensile Stresses

As a rule, economic considerations dictate that the walls of large panel buildings be designed with as little reinforcing as possible. Wall panels in the past have varied from totally unreinforced in lower structures which are less susceptible to unusual risks,<sup>(51)</sup> to moderately reinforced in taller buildings, especially in seismic areas.<sup>(51,53-56)</sup> Conflicting minimum reinforcement requirements have been suggested in a number of recommended practices<sup>(53,57)</sup> and building codes.<sup>(15,58,59)</sup> These vary from zero to a value of 0.2% of the gross area of concrete, both vertically and horizontally. Since wall panel reinforcement is considered in detail in the subsequent report on walls, the discussion here will be limited to minimum reinforcement requirements for the completed structure.

In tall structures tensile stresses often develop in wall assemblies under lateral load conditions; net tensile stresses due to bending can also occur. As a result, it is necessary to ensure tensile continuity between consecutive wall members. The building code requirements for structural plain concrete<sup>(58)</sup> allow net tensile stresses on the cross section of unreinforced concrete members. This is a safe specification only if the homogeneity of the concrete elements is assured throughout the structure. In a high-rise LP building it does not seem reasonable to assume that all wall panel sections will remain uncracked during fabrication, erection and the full service life of the structure. In addition, the connections at the floor level cannot transfer any tensile forces without reinforcing crossing the joints. Accordingly, it appears consistent with current practice to require that all net tensile

(Fig. 10):

- (a) compressive and tensile stresses at the longitudinal boundaries of the diaphragm,
- (b) shear stresses at the transverse boundaries of the diaphragm, and
- (c) shear stresses along the longitudinal and transverse edges of the floor plank.

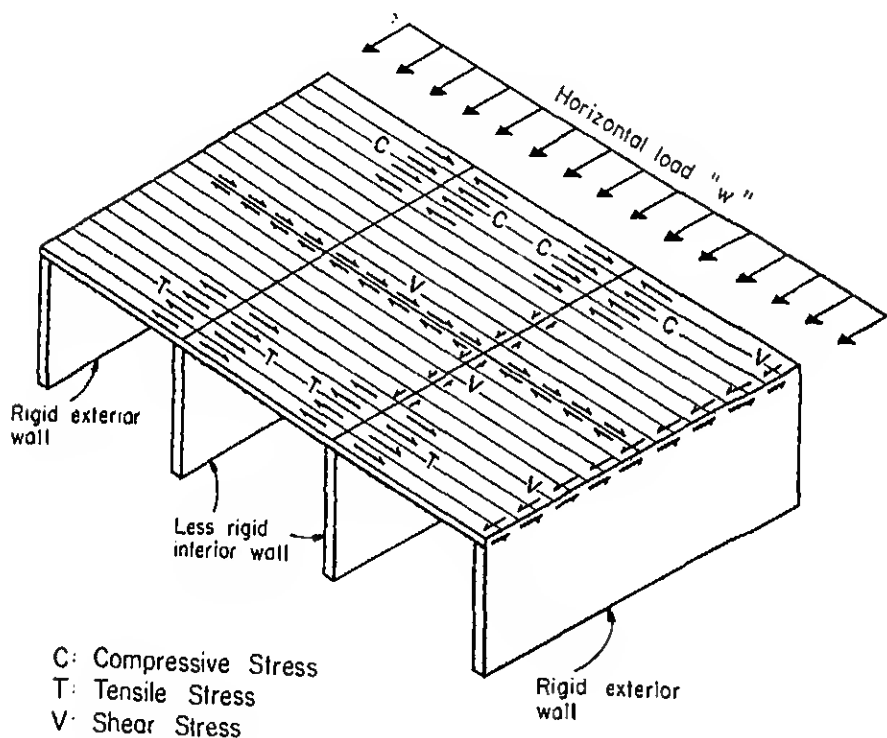


Fig. 10 Stresses in Horizontal Connections induced by Diaphragm

in combination with, the grouted keyways, mechanical connectors may be used. Alternatively, a structural cast-in-place topping properly bonded to the precast plank can act as a diaphragm.

### 2.2.7 Concluding Remarks

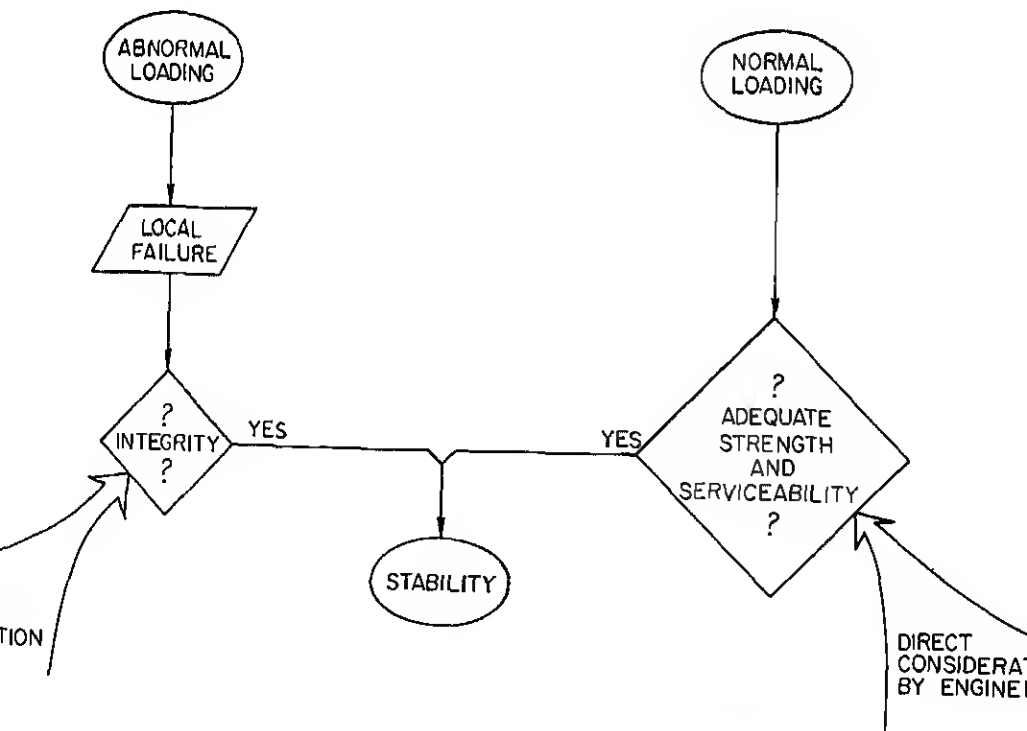
With few exceptions, methods and concepts used in the design of conventional structures are applicable to LP construction. However, to assure satisfactory performance in relation to both strength and serviceability, the structural engineer must pay particular attention to the effect of connection details on overall behavior.

Except under earthquake loading and severe wind loading, numerical analyses generally do not indicate a need for tying the components together. As a consequence, joints could be detailed with little or no continuity provided between members. The resulting structure would depend solely on friction and gravity for its stability. Such a structure, although designed for all code-specified loads, may, in fact, be extremely susceptible to total or partial collapse from a local failure caused by an abnormal loading condition. To assure the stability of the building under all loading conditions that may occur during its service life, the response of the building to abnormal loading conditions must also be considered.



phenomenon in which the spread of a local failure eventually results in the collapse of an entire building, or a disproportionately large part of it. Abnormal loading conditions and effects which can be structurally significant<sup>(1,61,62)</sup> include:

- service system (gas) explosions,
- explosive bombings,
- external explosions from accidents involving transportation of hazardous materials,
- ground vehicular collisions with buildings,
- aircraft collisions with buildings,
- tornados,
- flooding,
- foundation settlements, and
- errors in design and construction.



structural integrity which would provide a certain resistance to effects of abnormal loadings (Fig. 11). Ductility of members and connections would help to redistribute loads from overstressed sections. In extreme cases when local failure occurred, continuity between members would help provide an alternate path for the load. These beliefs were justified for some traditional forms of construction which have continuity and ductility; however, they were not valid for every form of construction. (66-74)

In 1968, attention of the engineering community was focused on the problem of lack of structural integrity in certain precast structures in a tragic way. In May of that year a progressive collapse of an apartment building occurred at Ronan Point in London (Figs. 12,13). A gas explosion in an apartment on the 18th floor caused an exterior panel to be blown out; this initiated a progressive collapse upwards to the roof and then almost down to the ground as debris fell on succeeding floors. Following the partial collapse of this large panel building, a public inquiry was held and a report describing the disaster was issued. (75) The inquiry determined that the design complied with all applicable building codes and that there were no deficiencies in workmanship. It was further determined that within the structural system, some elements resisted lateral forces solely by bond and friction.

This incident prompted further investigations into other collapses throughout the world. (76-78) These and other studies indicated that the problems associated with progressive collapse and abnormal loads concerned multistory construction of all forms, and were not limited strictly to large panel construction. (61,64,72-74,79-87) As a result of these investigations, two facts became evident:

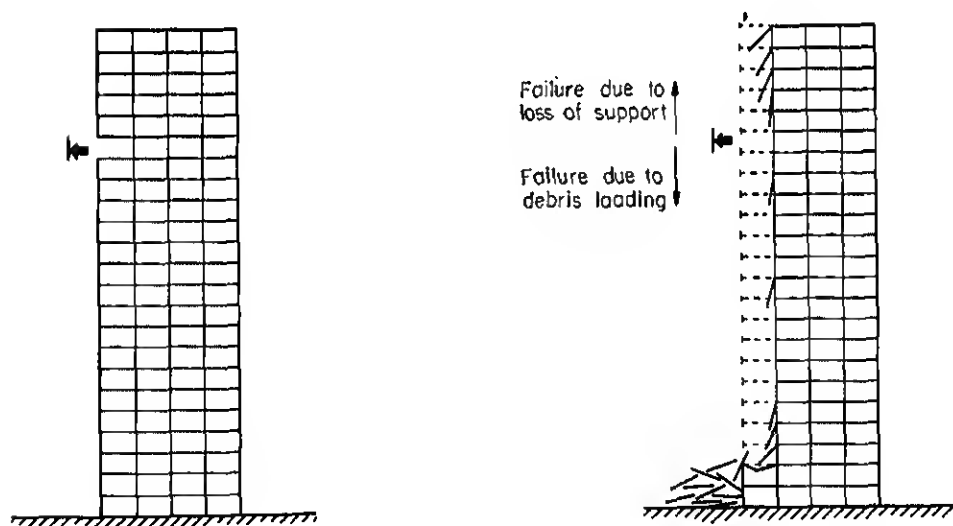


Fig. 13 Collapse at Ronan Poir

2. Resulting from insurmountable debris loading (e.g. the collapse of Ronan Point from the 18th story to the ground).

### 2.3.1 Reducing the Risk of Progressive Collapse

The explosion at Ronan Point initially caused only local damage that is, a wall element was blown out from the structure (Fig. 14a). In terms of hazard, the initial damage was of minor sequence. The progressive collapse which occurred (Fig. 14b) however, was the result of the inability of the structure to recover over the local failure, that is, its lack of integrity. Thus it is not the hazard of abnormal loading, but the susceptibility of a structure to progressive collapse which presents the real hazard (Fig. 15).



(a) Immediate Local Damage

(b) Progressive Collapse

Fig. 14 Failure Modes of Ronan Point Collapse  
(form Ref. 86c)

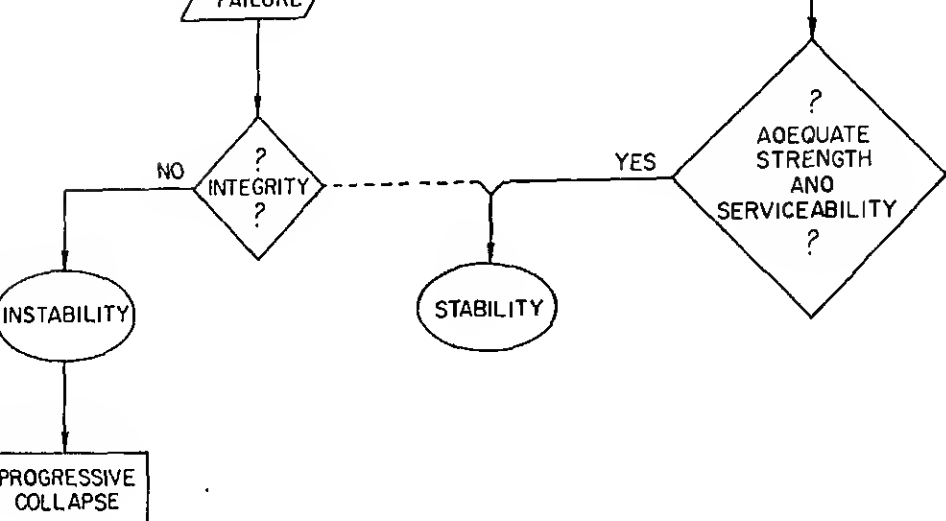


Fig. 15 Schematic Diagram of Design Process with Inadequate Detail Standards

Three possible approaches widely varying in concept can be employed to reduce the risk of progressive collapse:<sup>(88)</sup>

1. Eliminate the hazards which cause local failures (e.g., elimination of gas installations in multistory buildings, as has been done in France).
2. Design the structure so that the hazard does not cause any local failure (e.g., 5 psi gas explosion load requirement initiated in Great Britain).
3. Allow the local failure to occur but design the structure so that progressive collapse does not occur (ensure an alternate path for the load).

provision of venting to relieve explosions, use of shock absorbers for crash barriers against vehicular collision, prohibiting construction in flood plains, etc. In this way, the forces and their effects due to certain abnormal events can be reduced or eliminated, thus lowering the overall risk from abnormal events. With the above exceptions, however, abnormal loads can hardly be eliminated or controlled. Since any realistic solution must deal with all abnormal loading conditions to some extent, this method of eliminating the hazards is not considered an overall or complete solution.

#### 2.3.1.2 Local Resistance

In the second approach, providing local resistance, prescriptive loading requirements are stipulated in an effort to provide sufficient strength to resist a local failure. A logical approach for dealing with the infrequent occurrence of an abnormal loading condition would be the probabilistic method.<sup>(65,89,90)</sup> Although in theory the probabilistic method may provide the answer, many elements necessary for its development are still lacking. Once suitable probabilistic design codes are developed and implemented, abnormal loads will become a part of the overall safety considerations. However, implementation does not appear to be imminent.

In lieu of a probabilistic approach, a minimum design load of 5 psi for gas explosions could be used. However, this approach which provides resistance to one specific abnormal load has been challenged on a number of grounds. From an economic viewpoint, additional costs to prevent local failure under a

by complex methods, no practical analysis and design procedures are currently available.

It should not be construed from the above discussion that all abnormal loading conditions will always remain undefinable. Certainly loads imposed by earthquakes were not always specifiable. However, because of the high local risk, research was carried out and reasonable rational methods of determining design loads were developed and adopted within the various codes and standards. Similarly, tornado effects are currently being studied intensively<sup>(1)</sup>, due to the increasing level of risk imposed by such loadings. It is anticipated that the semiprobabilistic approach currently being developed for tornado loading will, when completed, be adopted and enforced, especially for the more critical building occupancies. With regard to any new "normal" loading conditions, it is necessary that they be applicable to all forms of construction, and not arbitrarily limited to those which may "appear" to be more susceptible to their effects.

### 2.3.1.3 Alternate Path through Structural Integrity

Since the nature and magnitude of most abnormal loads are unpredictable, the third approach, of allowing local failure to occur but providing an alternate path within the structure to avoid a progressive collapse, appears to be a sound concept. However, as a consequence, a limiting (maximum) damage volume or area must be defined in some way.

When a structure has the ability to bridge over local failure

structure to mobilize reserve strength needed to resist progressive collapse.

The General Structural Integrity approach can be implemented in two distinct ways: given a damage volume or area the engineer can be required to apply a rational procedure (using alternate path design) to establish the necessary integrity (direct design); or, alternatively, code writers can develop the necessary minimum detailing practice to establish a degree of continuity and ductility (indirect design). In either case it is necessary to identify the special sensitivity to abnormal loadings that the particular structural form possesses. (66-

Within this report minimum detailing is recommended as the most viable alternative.\* In this approach, specific criteria (in the form of tensile tie forces, deformability requirements, wall layout, etc.) are given in the appropriate codes and specifications as minimum requirements for the design and construction of LP structures. Although this indirect approach is by its nature more general, it has a number of advantages:

1. Code writers and researchers have a general responsibility to evaluate details to assure the safety of structures.
2. Design engineers should not be required to directly consider the effects of abnormal loads for one type of construction and not for another.

At present this approach has been adopted by the PCI Committee on



minimum detailing requirements based on sound engineering judgment can establish an adequate degree of structural integrity.

Presently most building codes do not contain provisions to establish GSI in large panel concrete structures. This shortcoming may be attributed to a combination of the following factors:

1. Building codes and standards have traditionally emphasized member design, while giving only limited guidance for the overall system design.
2. Design of connections has generally been omitted from earlier code specifications.
3. New forms of construction have been developed in which connections are extremely critical.

While by its very nature LP construction does not lend itself easily to a design with moment continuity, it is possible to develop force continuity and ductility of the connections as well as of the overall system. (69,82,83,91-95) It is recognized that minimum requirements cannot possibly encompass all conditions. Therefore, they should be supplemented with an educational program to familiarize the engineering profession with the problems encountered under abnormal conditions.

This Structural Integrity method, which has already been adopted in part by other countries, introduces a philosophy of design emphasizing strength and ductility of connections

The objective of the remainder of Section 2.3 is to present the concepts used in establishing minimum detailing criteria to provide continuity and ductility in the connections and structure. With the methodology established, experimental testing can then be performed to develop the necessary data which will assure adequate force-deformation characteristics for the connections. It is anticipated that the continuity details will be specified as recommended practice, while the force-deformation characteristics will function as performance criteria when judging the acceptability of new materials and construction methods. Specific recommendations for both will be given in subsequent reports.

### 2.3.2 Rationale for General Structural Integrity

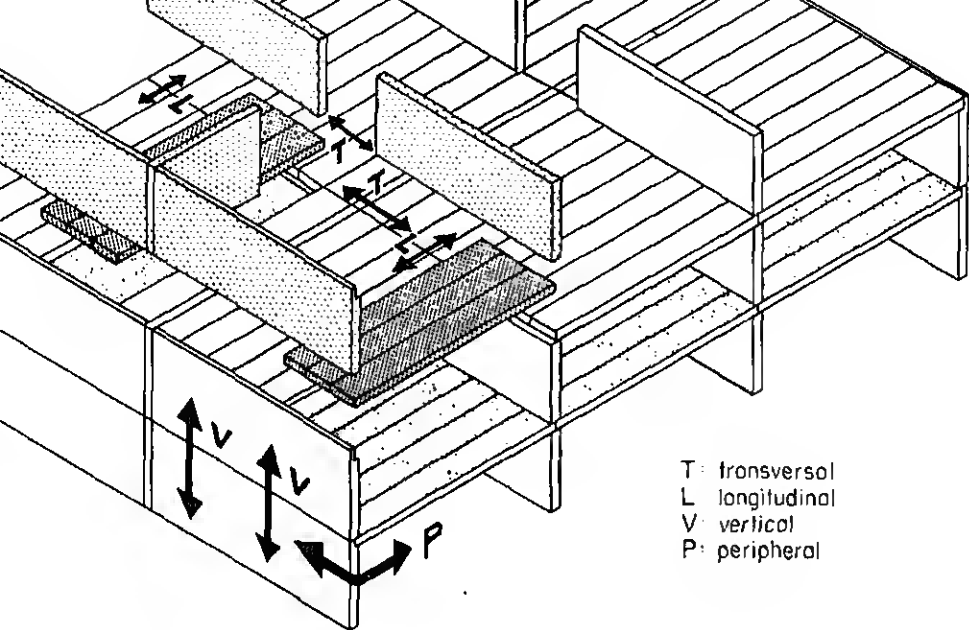
Connections between the precast elements of LP structures are recognized as the weak link in this structural system. This weakness is attributable to the lack of continuity through the connections, and to their brittle nature. Theoretically, friction-type connections can provide sufficient stability for gravity loads within an LP structure; however, such connections offer little resistance to the effects of abnormal loadings. Recent studies and discussions<sup>(69-74,78,97-101)</sup> on the performance of structures under normal and abnormal loads have reiterated that the integrity of a structure, regardless of the type of construction, depends on its connection characteristics; and that the strength of the elements cannot be utilized if the connections have insufficient strength. Therefore, as stated earlier, a degree of continuity across the connections and ductility within the connections should be ensured to achieve General Structural Integrity.

The magnitude of required continuity can be determined by assessing the forces acting within the connections under various conditions of local failure. Because of the nature of LP structures, these forces occur only as compression, tension, and shear.\* Adequate compression and shear capacity in LP connections will usually be provided by design under normal loads. However, little tensile capacity is found between elements in LP structures, and therefore tensile capacity between elements across the connections (both horizontally and vertically), must be provided. This required tensile continuity across and within the connections can be achieved by providing the following ties (Fig. 16):

- transversal,
- longitudinal,
- vertical, and
- peripheral.

To develop a rationale to determine qualitatively the function of the ties and the required magnitude of tie forces, a realistic model must be selected to assess structural behavior following local failure. Failure mechanisms within this model should incorporate post-elastic material behavior with large localized deformations. The basis for minimum detailing must encompass the following aspects:

Development of significant moment continuity between individual elements is not considered essential technically nor easily attainable economically in the usual American-type LP systems.



## 16 Schematic Location of Tensile Ties in a Large Panel Structure

1. Extent of damage--an assessment of the nature and extent of local damage that is likely to occur under abnormal loadings;
2. Alternate paths--evaluation of alternate structural actions which can develop as a consequence of a local failure to re-establish load flow in the remaining undamaged structure--stability analysis of the partially damaged structure; and
3. Tie requirements--quantitative determination of requirements for transversal, longitudinal, vertical and peripheral ties to bridge local failures and to assure stability of the partially damaged structure.

### 2.3.3 Damage from Abnormal Loads

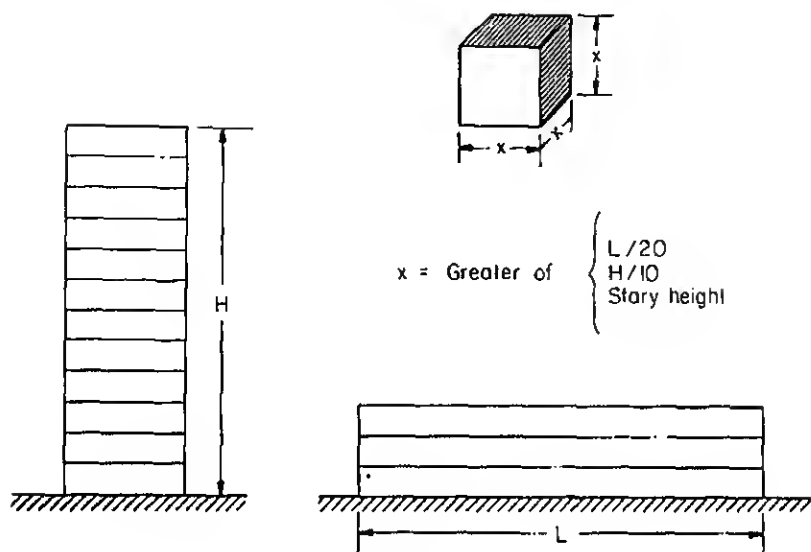
The indirect method of establishing General Structural Integrity in LP structures does not require the engineer to consider damage volumes (areas) or alternate paths in his design of the structure. However, to develop the minimum tie requirements to assure GSI, probable types and extents of damage must be defined. Reasonable definitions and limits of damage volume are suggested in this report only as a guide for a rational basis to determine tie requirements.

#### 2.3.3.1 Damage Characteristics

Code writing bodies in the United Kingdom<sup>(59)</sup> and Sweden<sup>(96)</sup> have acknowledged that a certain amount of local structural damage is considered inevitable and acceptable by specifying the extent of local failure which the structure must be able to bridge (Fig. 17). The specified amount of damaged load-bearing wall is assumed "notionally removed" from the structure at any location within a given building.

The main premise when using the "notional removal" approach is that an element or portion of the structure has been totally removed. In LP buildings with brittle joints which depend primarily on friction and bond under compressive loadings, the "notional removal" concept may provide the basis for a realistic representation of their response to abnormal loadings. However, with ductility and continuity in the form of vertical

# **SWEDEN: DAMAGE VOLUME**



## **UNITED KINGDOM: NOTIONAL REMOVAL OF ANY VERTICAL COMPONENT**

### g. 17 Damage Criteria

With LP structures tied together horizontally and vertically, it is more reasonable to assume that wall panels become "ineffective" as a result of abnormal loading, i.e., they will no longer function as load-bearing members as originally intended, but will remain in place in their damaged condition

Besides being considered more realistic, the concept of ineffective behavior is introduced because of its effect on debris loading and slab behavior in the damaged state. Specifically, the requirements for the slabs to "hang together

When estimating damage extent, the following must be taken into account:

1. The consequence of failure of a particular element in the structure;
2. The influence of a particular structural configuration and layout of the walls; and
3. Load magnitude, if known or can be reasonably estimated.

Load bearing wall panels are the primary load transfer element in an LP structure. These panels which generally range in length from 25 to 45 ft., are "apartment" size rather than "room" size as is typical in European and Scandinavian construction. Because of the large size of wall elements used in American construction, it is unlikely that most abnormal loads would effect an entire wall panel<sup>(1)</sup>. However, because walls are not generally braced at or between ends, it is reasonable to assume as an outside limit that an entire wall element located anywhere in the structure can become ineffective (Fig. 18). This somewhat stringent requirement is considered necessary due to the importance of the member in the structure.

This requirement can be relaxed to a degree if measures are taken in the design to ensure that only a part of the wall becomes ineffective. Such measures include stiffening elements in the form of wall segments or integral columns (Fig. 19). These stiffening elements however, must in turn be capable of

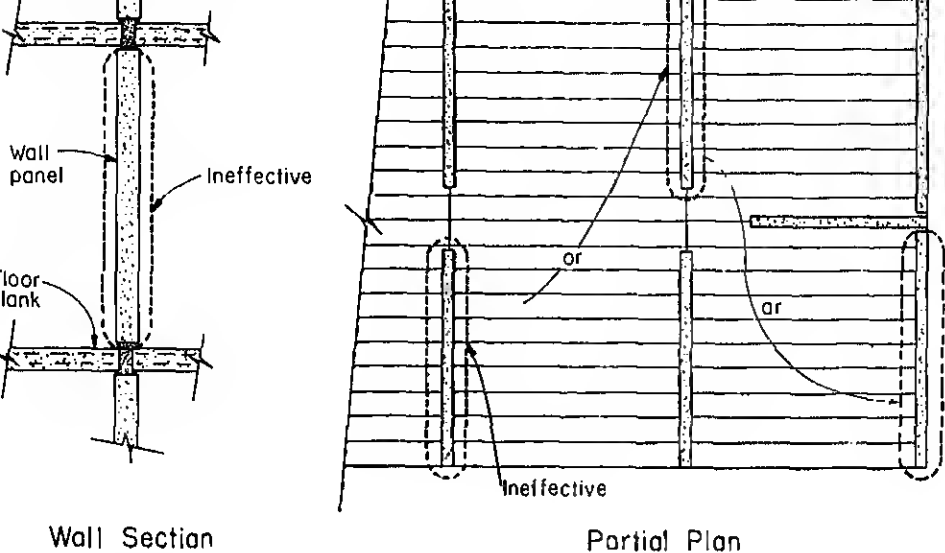
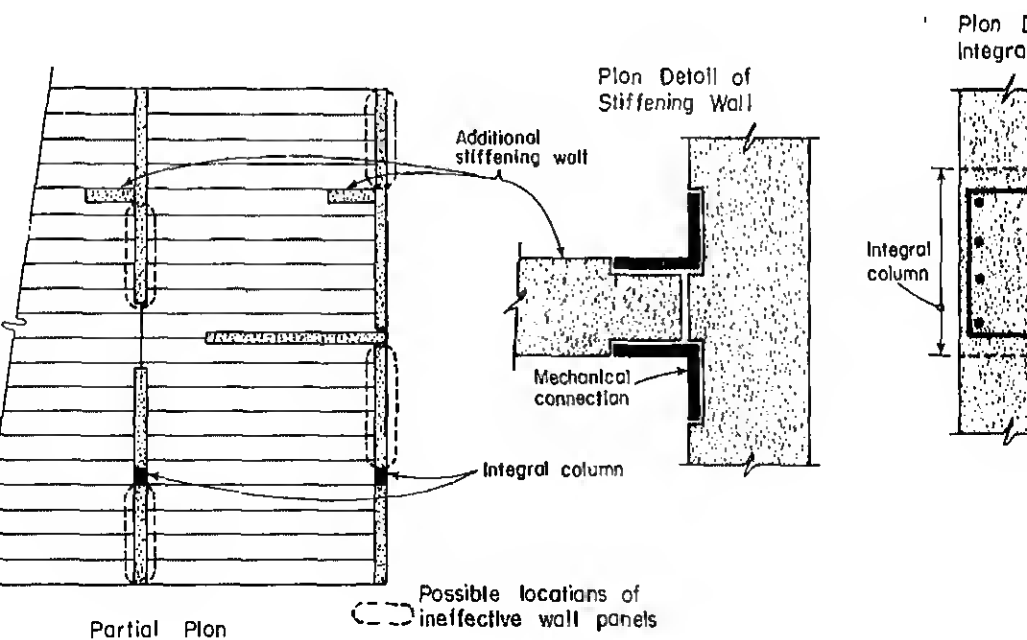


Fig. 18 Extent of Assumed Maximum Wall Damage Under Abnormal Loads





psi (based on a gas service system explosion) is suggested as a minimum design loading for these elements.

### 2.3.3.3 Damage in Floor/Roof Panels

Floor/roof panels, or plank, are secondary load-transfer elements as they support only their own weight and superimposed loads. Their role with regard to overall stability, of bracing the walls against buckling, is less critical than the function of the primary load-bearing wall elements. Nevertheless, the damage to slab elements and the consequence of that damage must be assessed and controlled.

Slab systems typically consist of side-by-side members (planks) up to 8 ft. wide. As a result, full continuity of the system perpendicular to the span is interrupted at each joint. It is reasonable to assume, therefore, that in case of an abnormal loading, only a few floor/roof planks would become ineffective for a full span length. As the slab consists of individual planks, there is limited tendency for the failure to spread beyond the planks affected by the abnormal loading. For slab elements, "ineffective" suggests that they are physically damaged and substantially weakened, and no longer capable of providing lateral support of wall units or of participating in the required diaphragm action. However, they are still strung together and capable of supporting, at least in part, their own dead loads within their original spans.

A more critical condition for slab panels occurs when they lose their stability due to partial loss of one of their end supports, i.e., a wall panel. The loss of such support would

giving the slab spans for the slab elements to hang together in their deflected shape. This catenary or suspension type action implies high ductility demands on the elements and that at right angles to the bearing walls. Such action is necessary to reduce impactive debris loading on the story below.

#### 2.3.3.4 Extent of Building Damage

Based on the above noted element ineffectiveness and assuming that only one abnormal load occurs at a time, an estimated extent of damage which the structure should sustain without further collapse can be established.

If slab elements become ineffective, the damage extent is limited to those floor/roof elements which have been impaired. If, however, a full size wall element becomes ineffective, the damage will extend into the story above to include the slab elements supported by the affected wall. Thus the maximum damage volume which the structure is expected to sustain would extend vertically two stories (where the wall is ineffective and the story above) and horizontally to the span on each side of the ineffective wall.

#### 2.3.4 Tie Functions and Alternate Paths

Structural design for "normal" loadings (those specified in code) begins with an assessment of the loads on the building. These loads should be safely transferred from the point of application to the final resisting point through a logical load path. If a part becomes ineffective as a consequence of an abnormal loading, a new load path must be established in the remaining undamaged structure, i.e., an alternate path must be created.

provisions should be made to effectively tie (or string) the steel elements together.

Tying the large panel structure together horizontally and vertically makes it possible to utilize the following structural mechanism to bridge local failures:

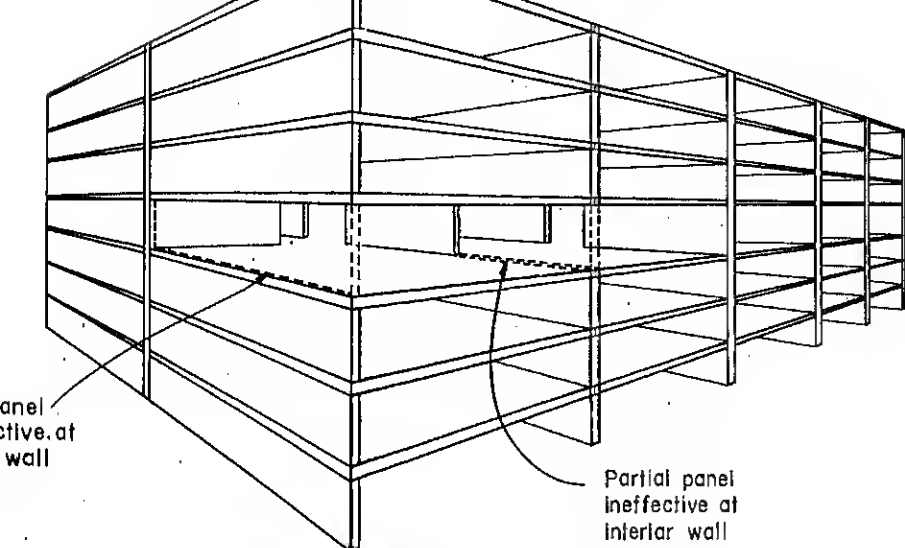
- (a) cantilever action of wall panels;
- (b) beam action of wall panels;
- (c) partial membrane action of successive spans of floor planks;
- (d) vertical suspension of wall panels; and
- (e) diaphragm action of the floor planks.

#### 2.3.4.1 Transverse Ties and Cantilever Action

When, at a given floor level, a wall panel or portion thereof becomes ineffective (Fig. 20), the wall panels above that level lose their support. The most effective method of transferring the loads is through cantilever action of the remaining wall panels above. However, to obtain cantilever action, the vertical load must be supported at the cantilever root.

This support can be provided by:

1. A vertical connection designed to carry the shear in each story in a wall assembly consisting of two or more adjacent vertical stacks (Fig. 21);
2. A vertically continuous return wall or a vertically continuous integral column at the interior edge of the wall assembly consisting of a single vertical stack (Fig. 22); or



## 20 Examples of Ineffective Panels

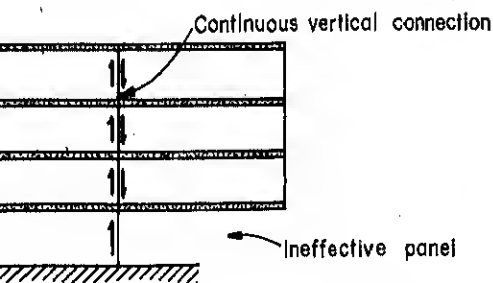
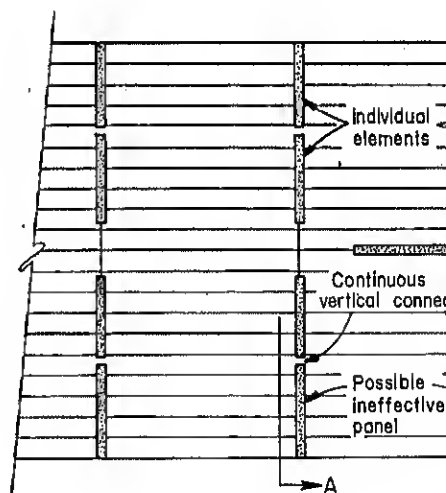
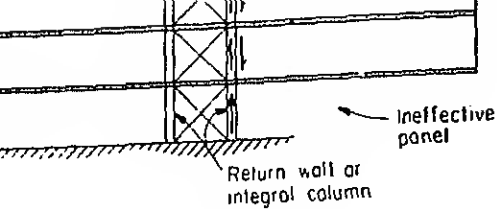


Figure A - Partial Wall Elevation

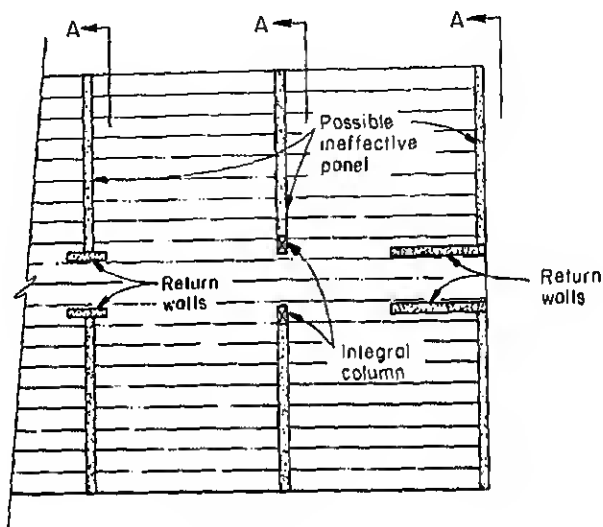


Partial Plan

## 21 Cantilever Mode - Two Adjacent Wall Panel Stacks



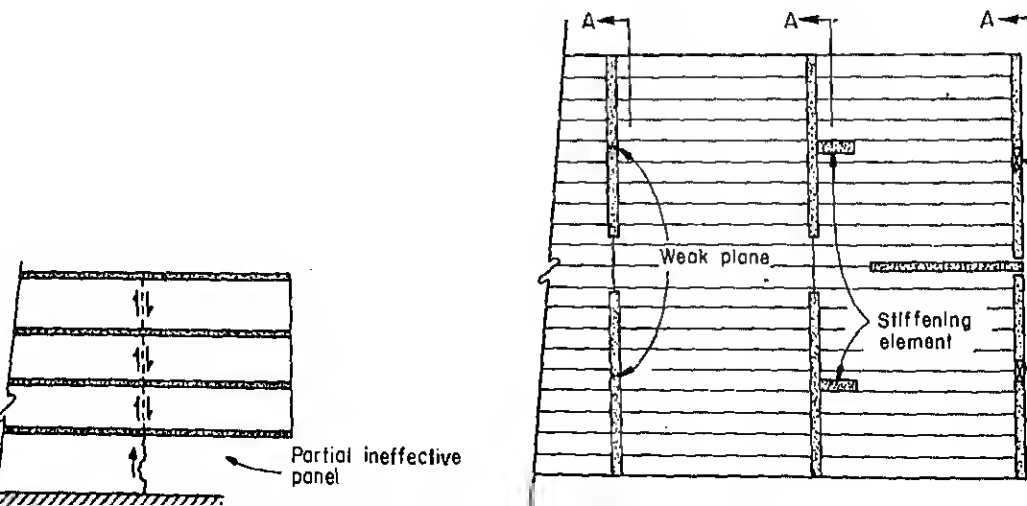
ion A - Wall Elevation



Partial Plan

Plan Section of Integral Column

## 22 Cantilever Mode - Single Vertical Wall Panel Stacks



continue across all vertical joints between adjacent panels. In addition, adequate resistance against overturning of the cantilever assembly must be ensured.

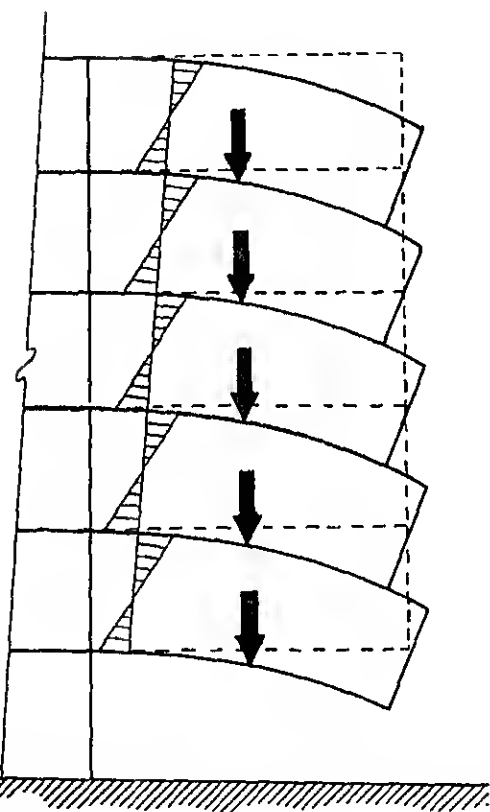
To assess the transverse tie force requirements and to develop details to assure cantilever behavior, the following need to be considered:

- (a) ability to transfer the tensile force of the horizontal tie at each story level into the wall panels immediately below;
- (b) shear characteristics of the interface between the top of the wall and the bottom of the slab;
- (c) shear characteristics of the interface between the top of the slab and the bottom of the wall at the horizontal joints between successive stories;
- (d) shear characteristics of the vertical connection providing cantilever support;
- (e) shear capacity of the wall panels; and
- (f) foundation-panel connection.

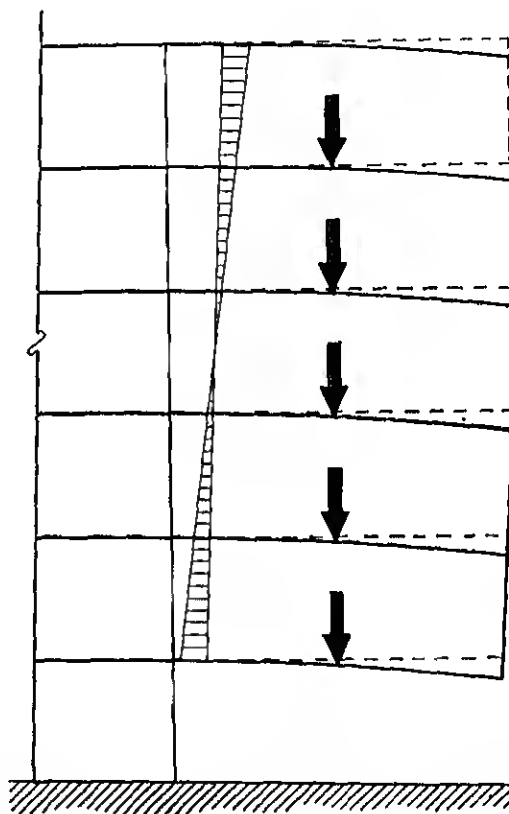
Of the above, shear characteristics of the horizontal interfaces both above and below the slab, and the shear capacity of the vertical connections, affect the behavior of the cantilever most significantly.

Depending on the horizontal connection properties, two modes of cantilever action can be defined as giving upper and lower bounds to the transverse tie force requirements. In the free cantilever action shown in Fig. 24a, no horizontal shear is transmitted between story-high cantilevers. Thus, they are free to slide past each other on the interface between the

sive wall panels. This provides shear resistance across the connections by shear friction, and consequently, enhance the cantilever strength. This behavioral mode provides a lower bound in predicting cantilever strength.



(a) No Shear Capacity in Horizontal Connections



(b) Adequate Shear Capacity in Connections

Fig. 24 Modes of Cantilever Action

most likely occur within the vertical tie connection details. Therefore, this behavioral mode should be considered an upper bound in predicting the cantilever strength.

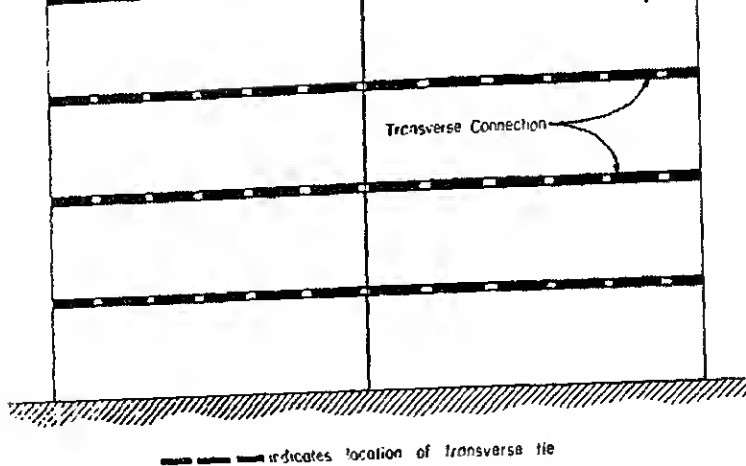
In practice the behavior will be somewhere between the two modes described above, and the tensile force requirements for the transverse tie in a multistory cantilever will depend primarily on the details affecting shear characteristics of the horizontal connections. Based on experimental tests to evaluate cantilever action, requirements for transversal ties and for performance and acceptance criteria for the horizontal connections will be given in a subsequent report on the analysis, design and acceptance criteria of connections.

The transverse tie within the connection can be in the form of mild steel or unstressed prestressing strand. To ensure its effectiveness, the tie should extend the full width of the structure, and be adequately secured to the peripheral tie system (Fig. 25).

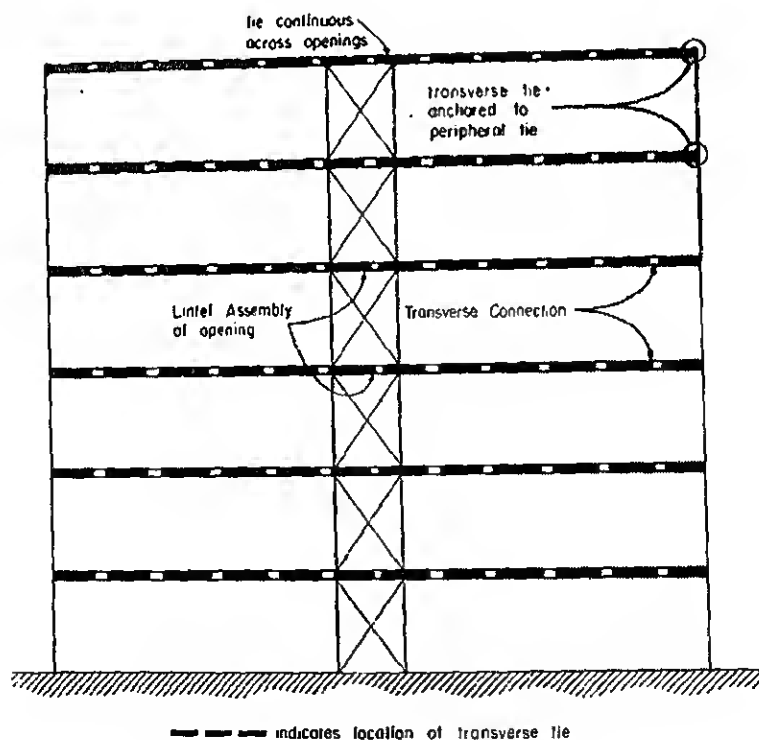
#### 2.3.4.2 Transverse Ties and Beam Action

Although cantilever action through the use of transverse ties is the single most important element in establishing alternate paths in LP buildings, in some instances of local damage the transverse ties can also be very effective in developing "beam" action of the wall panels. However, to develop beam action, a capacity to transfer the vertical load to each of the supported ends must exist. Such supports are available in the cases shown in Fig. 26.





### Flank Wall



### Interior Wall

Fig. 25 Suggested Extent and Location of Transverse Ties

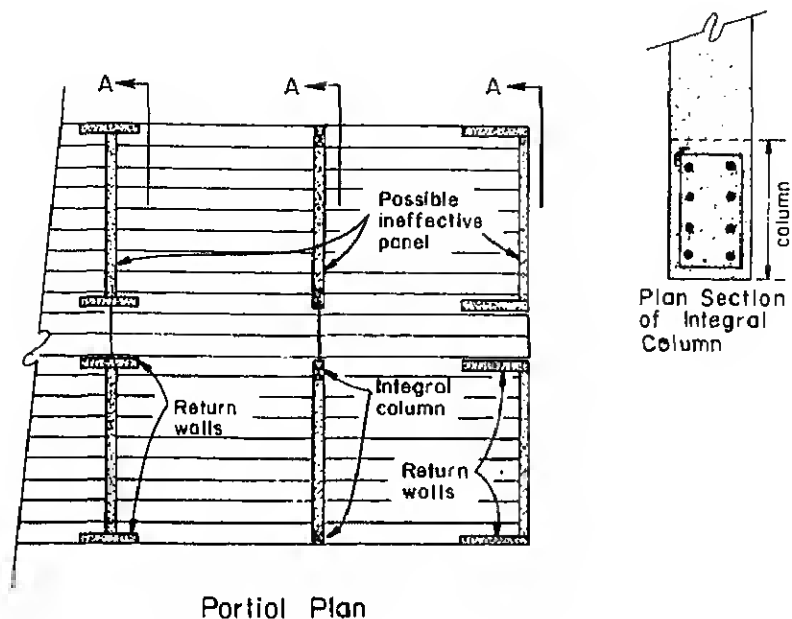
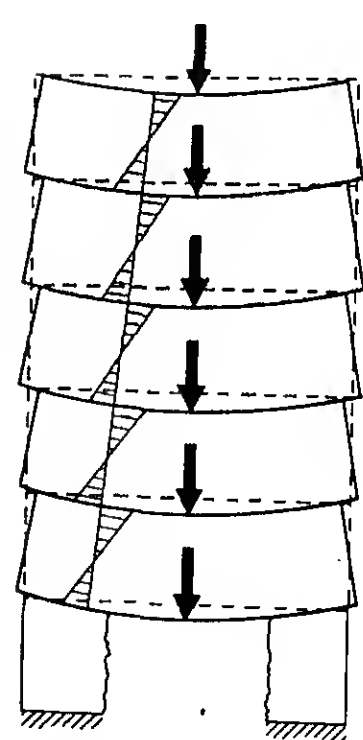


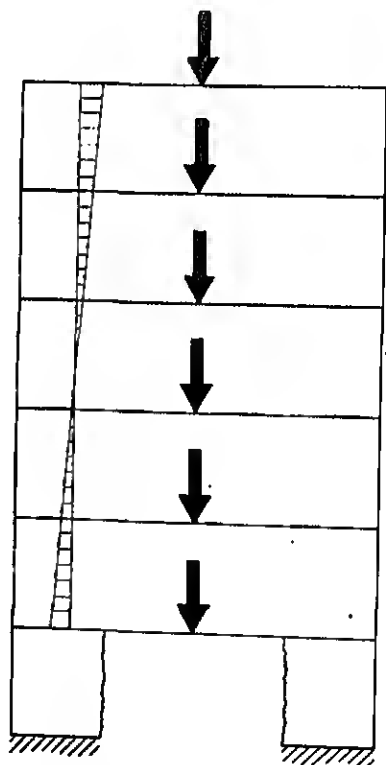
Fig. 26 Beam Mode - Vertical Wall Panel Stacks

As in the case of cantilever action, the vertical su  
 beam action are provided through proper panel design  
 layout. The beam's flexural resistance can be devel  
 if adequate tensile capacity in the form of reinforc  
 exists in the transverse horizontal connection or wi  
 wall panels themselves. Again this continuity would  
 across all vertical joints between adjacent panels.  
 factors affecting the overall behavior of this alter  
 flow are similar to those examined under cantilever  
 Vertical load-carrying capacity at each end must be

27a, the horizontal connections offer no resistance against relative slip between the consecutive story-high wall panels. Thus, the wall panels slide past each other, and the total strength of the beam is the sum of the flexural strengths of the individual wall panels. Under these conditions, rather than transverse ties, tensile reinforcement can be embedded in the panel itself. This differs from the case of cantilever action where the tensile capacity must be provided by the transversal ties embedded in the horizontal connections.



(a) No Shear Capacity in Horizontal Connection



(b) Adequate Shear Capacity in Horizontal Connection

Fig. 27 Modes of Beam Action

tions offer full resistance against slippage and consequently the beam acts as a monolithic element. This behavioral mode is an upper bound of the beam strength.

The necessary ties to resist the tensile forces from the bending moment will be less than those required under similar span cantilever action; however, this structural behavior will usually require more return walls or integral columns to assure vertical supports. The transverse ties to effect beam action can be either of mild steel reinforcing or unstressed prestressing strand. The ties should be continuous the full width of the building and anchored to the peripheral tie system (Fig. 25).

#### 2.3.4.3 Longitudinal Ties and Partial Membrane Action (Large Deflection of Slabs)

Catenary action of the continuous slab spans due to loss of supports has been investigated in Europe. (82,94,102-104) Although full catenary action is accepted as an appropriate and functional means of re-establishing the load flow in European LP structures, it is not considered in this study a suitable method for American LP construction for the following reasons:

1. The typical slabs for European systems span from 10 to 18 ft., are cast for a particular building, and are joined together by interlacing loops protruding from the slabs. LP systems used in the United States have floor spans which range from 20 to 40 plus feet. The U.S. slab systems are generally

necessary (slab span) is about twice that of its European counterpart, the problems related to physical development of the necessary longitudinal tensile forces are greatly magnified.

2. To develop the necessary catenary action at required force levels, the floor systems must undergo large deflections. With 40 to 80 foot catenary spans in U.S. construction, the necessary deflection approaches a total story height. However, to develop alternative paths through story high deflections of slabs is simply not acceptable due to serviceability considerations for the structure in the damaged state.

The major element of resistance and stability in the damaged wall-type structure is provided by cantilever and beam action. Therefore, should an interior wall become ineffective, the only function of the slab is to ensure a stringing together of its elements to inhibit progressive collapse from debris loading. This is necessary since the slabs are generally not tied directly into the wall above.

To string the slab elements together in a deflected shape (Fig. 28), tensile continuity must exist between them, above the ineffective wall. Ductility of the connection must be insured to satisfy the large deformation demand and resist the dynamic effects that may occur. In a catenary structure of this type the deformation (or ductility) increases with increased deflection while the tie force decreases.

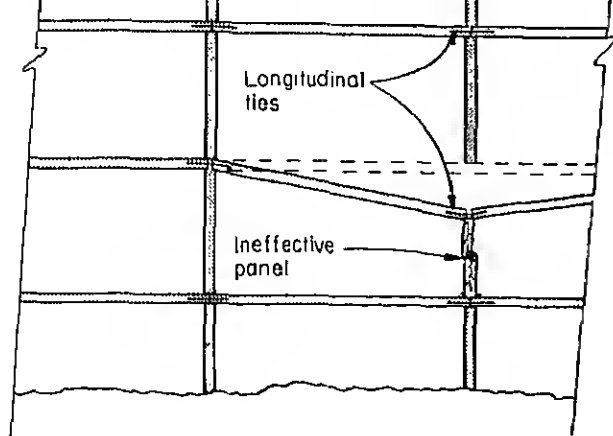


Fig. 28 Partial Membrane Action of Floor Elements at Ineffective Panel

Continuity can be developed with reinforcing placed in the longitudinal horizontal keyways between floor panels, or in the topping slab if one is employed. To ensure integral behavior, the ties of the end span should be anchored into the peripheral tie system. Since floor elements are well reinforced, the ties need not be continuous for the full length; the tie force should be transferred by bond or mechanically from an element in one span to an element in the next span (Fig. 29).

The elongation in the plane of the slab required to achieve the large deflection at the ineffective support must take place primarily in the ties at the supports. Parameters affecting the elongation characteristics and the overall effectiveness of the tie within the joint include:

- (a) keyway shape and size affecting its longitudinal shear capacity;

parameters on the overall strength and ductility of the longitudinal ties will be evaluated. The tests have a two-fold objective: (a) to form the basis for establishing minimum force requirements and performance criteria for various key joint configurations presently in use, and (b) to develop acceptance criteria for newly developed details. The tie requirements and criteria will be examined in detail in a subsequent report.

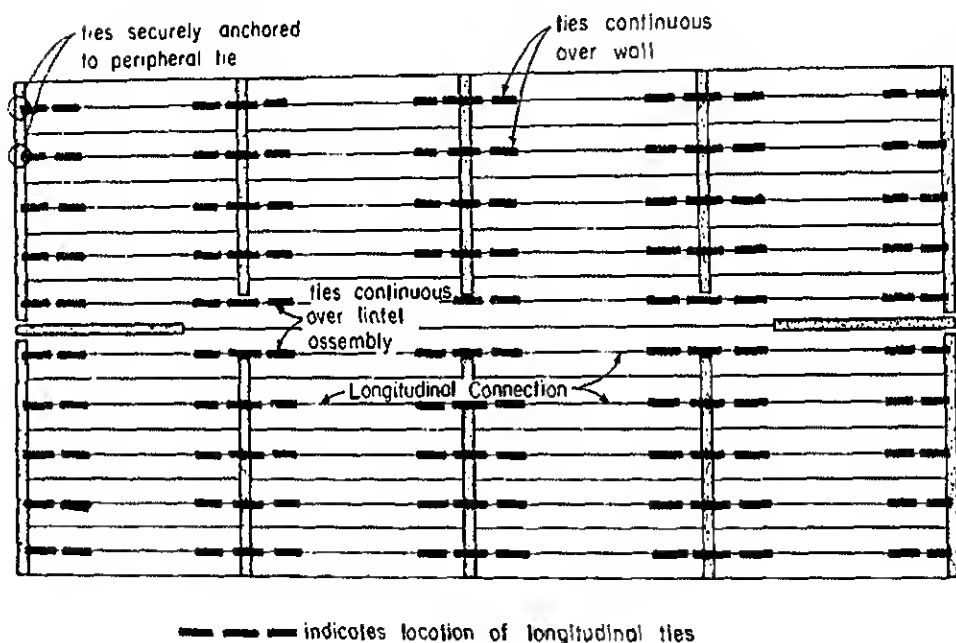


Fig. 29 Suggested Extent and Location of Longitudinal Ties in Floor Roof Panel Assembly

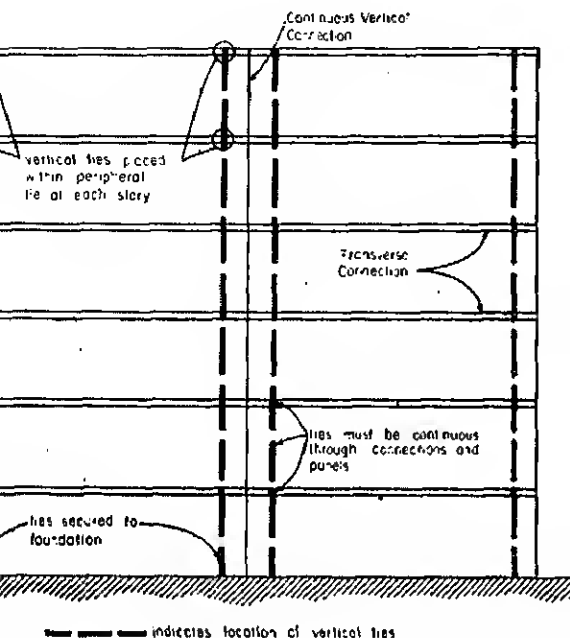
1. Provide vertical suspension of ineffective wall panels to limit debris loading.
2. Provide resistance against "kicking out" of the walls sideways, thus fostering "ineffective" behavior of wall elements rather than total removal under abnormal loading.
3. Assure clamping and dowel action in the horizontal connection for shear friction to resist horizontal shear and thus develop cantilever and beam action.

If an alternate path for the load transfer around an ineffective wall panel has been established either through cantilever or beam action, the damaged panel should be suspended from the alternate supporting mechanism. To ensure this suspending action, vertical continuity in the form of tensile ties must exist between and through all wall panels. Tensile continuity from foundation to the roof can be assured by providing vertical ties proportioned to resist (within each story) the dead loads of the wall plus those loads superimposed by the floor panels. In addition, the vertical tie must resist the stresses from shear friction clamping in the horizontal connection, as discussed in Sections 2.3.4.1 and 2.3.4.2.

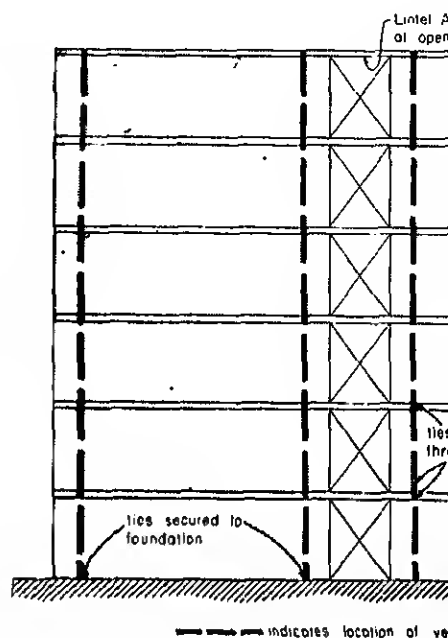
Because of the increased vulnerability of exterior wall panels (as compared to interior wall panels) with regard to abnormal loadings, greater vertical continuity is desirable in flank wall assemblies in order to afford them increased protection against removal from the structure. The vertical tie of the flank walls should be placed within the peripheral



(Fig. 30). The ties can also function as reinforcement required for service and erection loads.



Wall



Interior Wall

## 0 Suggested Extent and Location of Vertical Ties

### 2.3.4.5 Peripheral Ties and Diaphragm Action

Peripheral ties in the damaged building should:

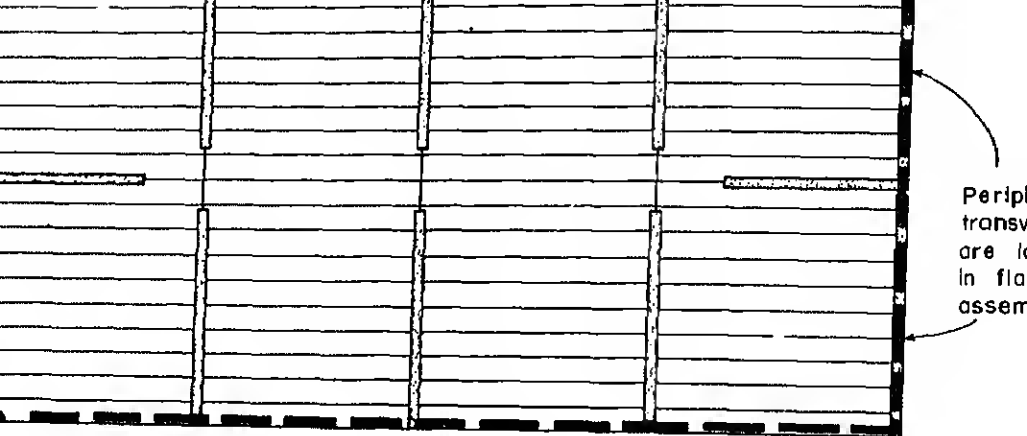
1. Establish the necessary diaphragm action to resist the effects of wind, torsion and unequal load distribution throughout the structure;

These functions can be accomplished by employing a ring beam at the periphery of the structure at each floor and roof level.

To assess the force level required in such a ring beam in a structure is a difficult, if not impossible, task. In the damaged state the entire slab or roof may act as a membrane, with its periphery subject to large ring beam forces. It is suggested that the peripheral tie forces be based on the diaphragm action necessary to resist normal lateral loads, assuming the customary load factors and capacity reduction factors. However, for purposes of continuity and anchorage requirements in the damaged state, the transversal tie forces required in the flank wall assemblies should be continued in the longitudinal faces (if larger than those required for diaphragm action) for the design of the complete periphery (Fig. 31). These continuous ties, which should be located as close to the actual periphery as possible, may be of mild steel or prestressing strand (Fig. 32a,b), or mechanical connectors between ends of planks in adjacent spans (Fig. 32c,d).

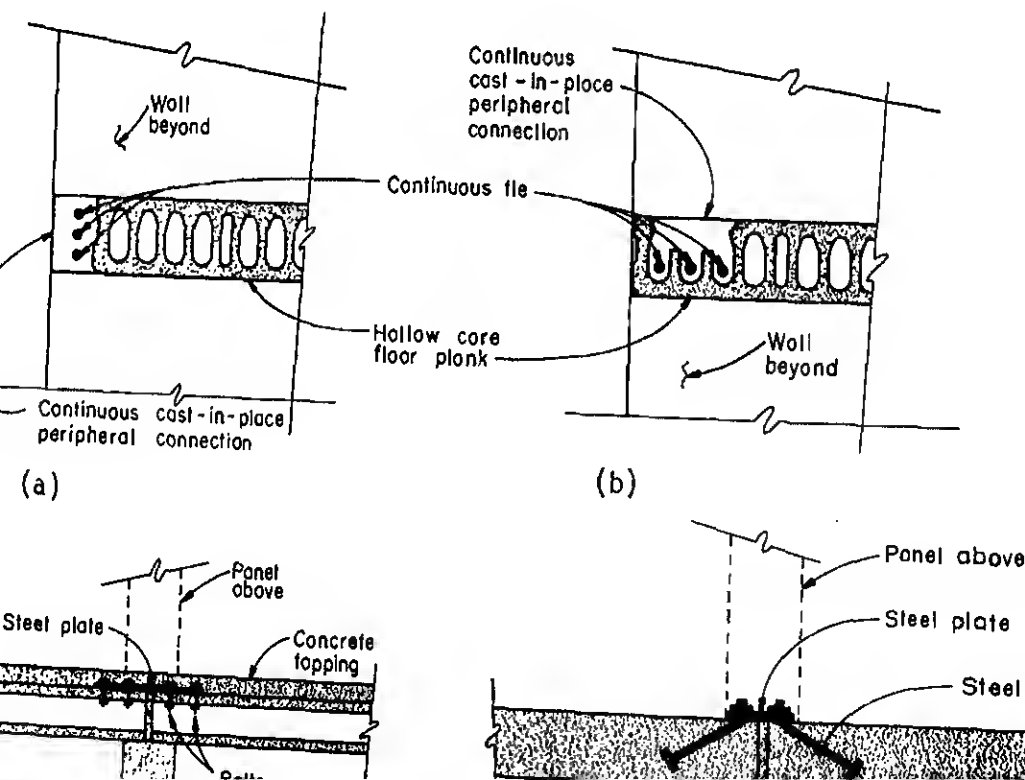
### 2.3.5 Debris Loading

Although ties are provided to hold the elements together, when a floor/roof panel is subjected to an abnormal load, the possibility of impact loading from spalled fragments still remains (Fig. 33). The magnitude and nature of such debris loading depends upon the mass and velocity of the fragments, as well as their shape. These variables in turn are functions of the modes of collapse of the



--- indicates location of peripheral tie

### 31 Suggested Peripheral Tie in Typical Floor/Roof Panel Assembly



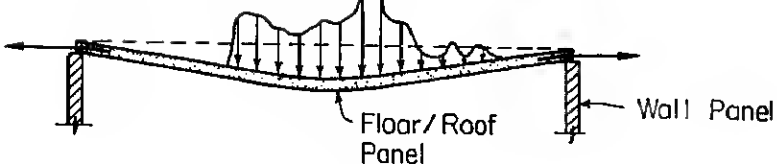
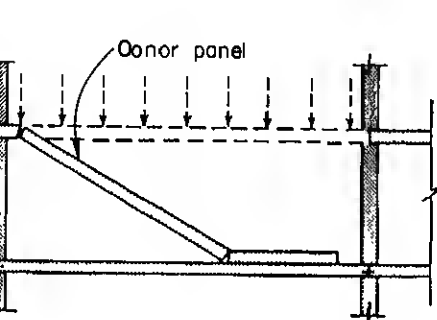
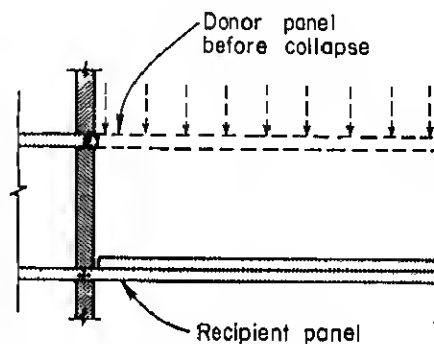


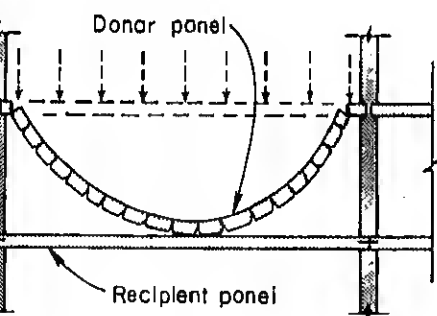
Fig. 33 Floor/Roof Panel Subject to Debris Loading



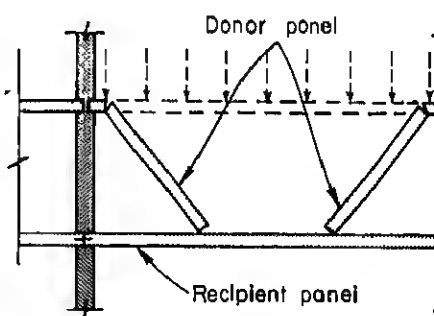
Loss of One Support



Loss of Both Supports



Deflection at Midspan



Fracture at Midspan

### 34 Modes of Floor/Roof Panel Collapse

For convenience, the slab panel which causes debris loading has been termed a "donor" panel, and the panel which is subject to the debris loading has been termed a "recipient" panel.

earlier. Further, the connection details developed, which will enhance the ductility for all connections, will be specified as recommended practice. As a result, except for situations, it seems unnecessary to consider debris loading in the design process.

### 2.3.6 Loads and Safety Factors in the Damaged Structure

To determine the quantity of ties needed for stability of the damaged structure, the necessary loads to be resisted must be established. Consideration should be given only to those loads likely to occur before temporary or permanent measures are taken to repair the damaged area. Included in this category are the dead load and part of the design wind and live loads.

Load-frequency distribution curves for both live and wind loads are time dependent. As a result, reduced live and wind loading is suggested, based on the supposition that the building will remain in the unsupported damaged state for a limited period of time. Load values ranging from 10 to 50% could be recommended depending on the assumptions made. In the absence of a definitive basis, a value of 1/3 the specified live and wind load is arbitrarily suggested.

Since the rate of occurrence of local damage due to abnormal loads is relatively low, and since the intent is to assure the stability of partially damaged structures, the above loads should be resisted at a level close to the ultimate strength of the structure, say 95% of ultimate capacity. Therefore, for the determination of minimum tie forces, liberal load factors,  $\gamma$ , are appropriate. However, the capacity reduction factors,  $\phi$ , which account

Combining the loads (D, L, W) and load factor,  $\gamma$ , the required strength equation may be expressed as:

$$U = 1.05 (D + L/3 + W/3) \quad (\text{Eq. 2.3})$$

Incorporating the capacity reduction factor,  $\phi$ , the safety factor of the damaged structure becomes:

$$\text{S.F.} = \frac{\gamma}{\phi} = \frac{1.05}{0.9} = 1.17 \quad (\text{Eq. 2.4})$$

### 2.3.7 Specifying Tie Forces - Unstressed Strand

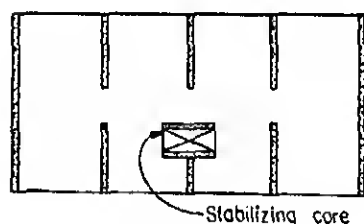
Specifying tie requirements in terms of forces rather than amount of steel permits flexibility in the choice of tie type, while fostering a better understanding of the technical issues involved. Specifying forces allows the use of unstressed prestressing strand which has the advantage of large tensile capacity with a relatively small diameter. Flexible strand also offers the advantage of easy placement in misaligned joints where a normal mild steel bar cannot be placed due to its rigidity. While unstressed prestressing strand cannot be used for resistance at service load levels since it would result in unacceptable deflections and crack widths, the large strains of the strand are not objectionable under catastrophic conditions.

### 2.3.8 Plan Layout

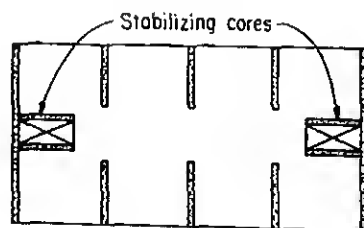
Proper structural planform, i.e., the layout of the structural walls, improves the overall rigidity and stability of the structure thereby increasing its structural integrity.<sup>(105)</sup> Effective structural layout is best accomplished by arranging walls in

tion. They will also improve the stability of individual transverse walls, while decreasing the length of wall likely to be affected by an abnormal load.

For exterior walls, returns or integral columns are desirable to improve their resistance to abnormal loadings. If vertical stabilizing cores are the only elements for lateral stability, it is desirable to distribute them throughout the building (Fig. 35) so that if one is damaged, total stability of the structure under lateral loads is not forfeited. Shrinkage, temperature and distribution steel placed in the topping can also be used to enable the slab to span in a perpendicular direction and prevent collapse where a support configuration is favorable for two-way action (Fig. 36).



Longitudinal  
stabilizing core  
centrally located  
within building



Longitudinal  
stabilizing cores  
distributed  
throughout building

Fig. 35 Possible Core Layouts in Typical Cross Wall Structure

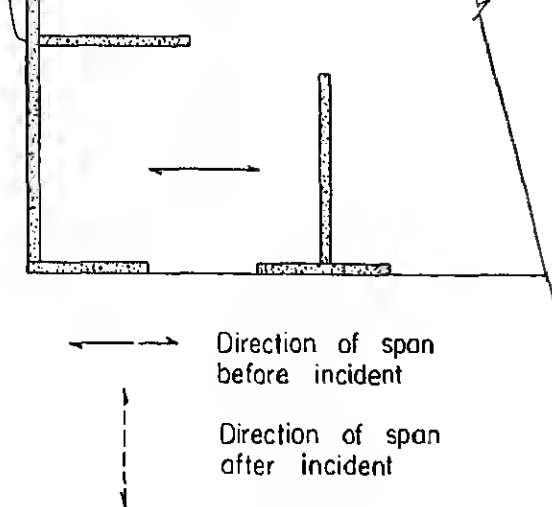


Fig. 36 Change of Slab Span Direction Under the Effects of Abnormal Loadings

### 2.3.9 Concluding Remarks

It is technically very difficult and economically prohibitive to design residential-type buildings for absolute safety. On the other hand, there is no justification for constructing buildings which do not afford a degree of safety with regard to abnormal loads.

To reduce the risk of progressive collapse of large panel residential buildings, a philosophy is developed to assure bridging of local damage while maintaining stability of the partially damaged structure by tying the components of large panel concrete structures together horizontally and vertically. Based on this philosophy, explicit requirements for a minimum tie system of transversal, longitudinal, vertical and peripheral ties will be developed.



to design for any particular abnormal load, since one allowing bridge local damage is provided.

In establishing structural integrity, the need for tensile continuity and ductility of the elements and their connections, as as of the overall structure, is recognized. This will be accomplished through a rational arrangement of tensile ties (Figs. 37,38) and through connection details which will assure an alternate structural path in the damaged building. This combination system continuity and ductility should enable the structure to either absorb the abnormal loads with minimal damage, or bridge localized damage as a result of the abnormal load. The provis

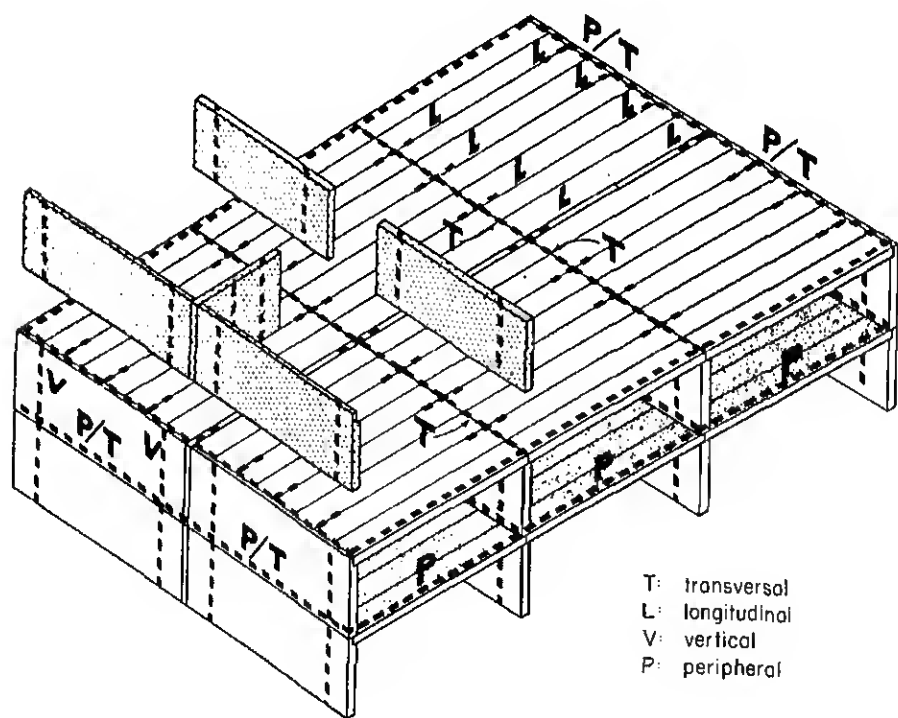


Fig. 37 Suggested System of Tensile Ties in Large Panel Structure

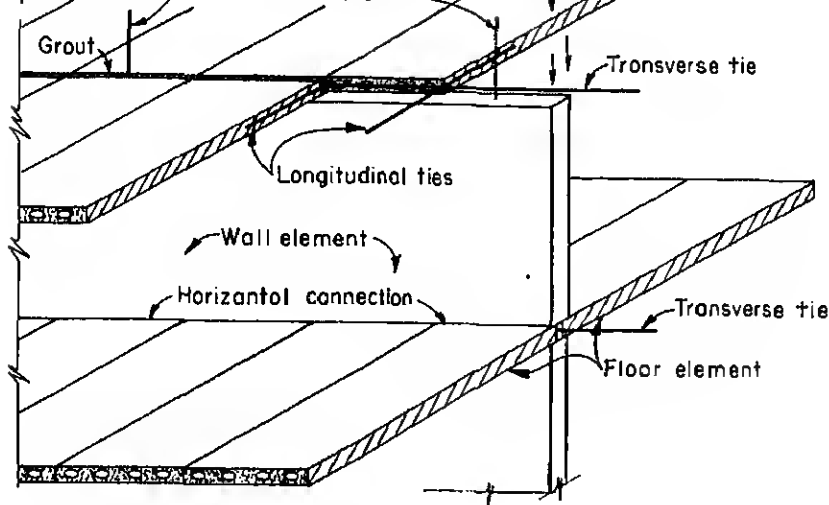


Fig. 38 Ties at Interior Wall-to-Floor Connection

of General Structural Integrity will bring the safety of LP structures closer to that of the traditional cast-in-place reinforced concrete buildings.

The objective of this approach is to limit and substantially reduce the general risk of collapse, as compared to that existing if no such measures were taken. It is not intended to afford absolute safety in regard to any exceptional event in any part of every building. Based on experimental tests performed thus far, it appears that the amount of ties required to achieve the desired level of General Structural Integrity will have a minimal effect on the economics of LP structures. With the philosophy established by subsequent experimental tests and analytical studies will focus on the development of purposeful details to optimize the effectiveness of the ties in the structure.

late the allowable deviation or "tolerance" from a specified dimension. A reasonable system of tolerances is an economic and technical necessity for the success of any precast system; however, despite their vital importance, tolerances have frequently been neglected. (52,56,106-109)

Tolerances are determined by the requirements of the entire construction process considering technology and economics of currently used methods of precasting, setting-out and assembly. The most influential factor affecting tolerances appears to be the connection type--more accuracy required for bolted connections than for grouted connections. Tolerances should guarantee correct assembly and efficient functioning of individual precast panels. Each panel is positioned within the basic space allotted it and should not encroach on the space allotted to another panel. Therefore, it is only necessary to fabricate and erect P.P. structures accurately enough to assure that the deviations do not fall outside known and acceptable limits. (107,108,110)

Little information is available on the cost of achieving various degrees of accuracy, though it is generally accepted that cost increases with accuracy. Most recommended practices (26,53,111-115) establish the permissible dimensional deviations in individual precast panels and in the panel assembly, and leave the methods for achieving accuracy to the discretion of the fabricator and erector. No recommendations as to how tolerances should be accounted for in design are available.

Requirements for dimensional accuracy in the design of connections, for assembly of panels, and architectural details have contributed to excessively small specified tolerances. As a result some current standards have been criticized as illusory, stringent, and vague, (56) and consequently, independent tolerance systems are in use. (56)

Panel deviations are caused by ambient temperature, moisture and stresses in the panels. Such deviations can occur during or after manufacture of the panel and should be taken into account when measuring a panel for conformity with the plans. These deviations are generally small in magnitude and their computations straightforward.

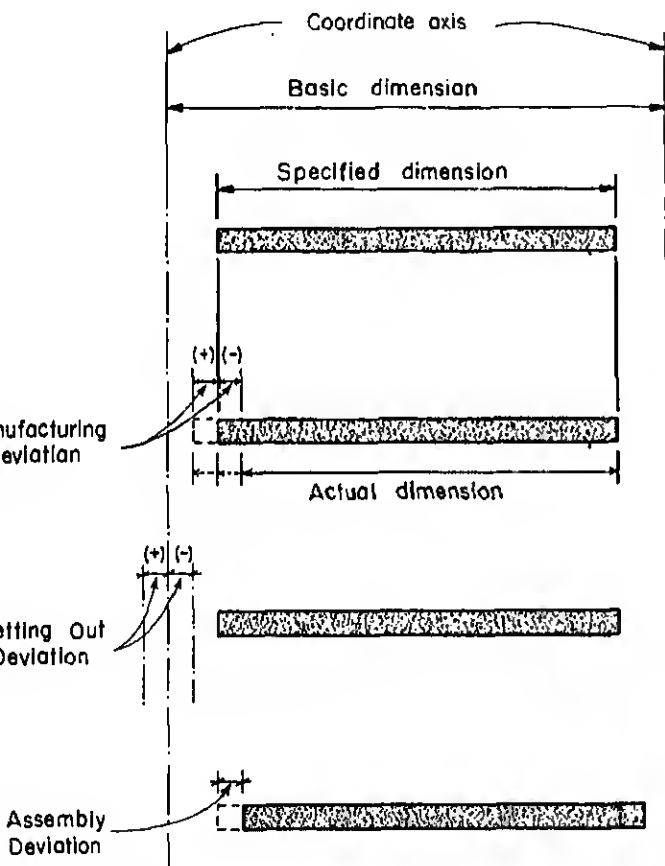


Fig. 39 Panel Size Deviations

in practice these deviations or variations are caused by human error as well as the limits imposed by measuring instruments and techniques.

#### 2.4.1 Manufacturing Deviation

Three types of dimensions of precast panels are shown in Fig. 39, namely:

- (a) basic or coordinating,
- (b) working or specified, and
- (c) actual dimensions.

Basic dimension denotes the dimension between the axes defined by the two-dimensional grid (a two-dimensional coordinate system of reference lines defining the layout of the LP structure). The working dimension is the planned dimension of the panel arrived at from both its basic dimension and joint dimensions. Actual dimension refers to the measured panel dimension after production. It differs from the working dimension due to manufacturing and material deviations.

Manufacturing deviations are caused by production method, equipment, formwork (mold), workmanship, and quality control. At present wall panels are cast vertically or horizontally in molds while hollow-core floor plank is either extruded or cast by special processes. The deviations which occur can be in each of the panel's three coordinate axes. Additional deviations that may affect elements include angularity, bowing (concave or convex), and warping (Fig. 40).

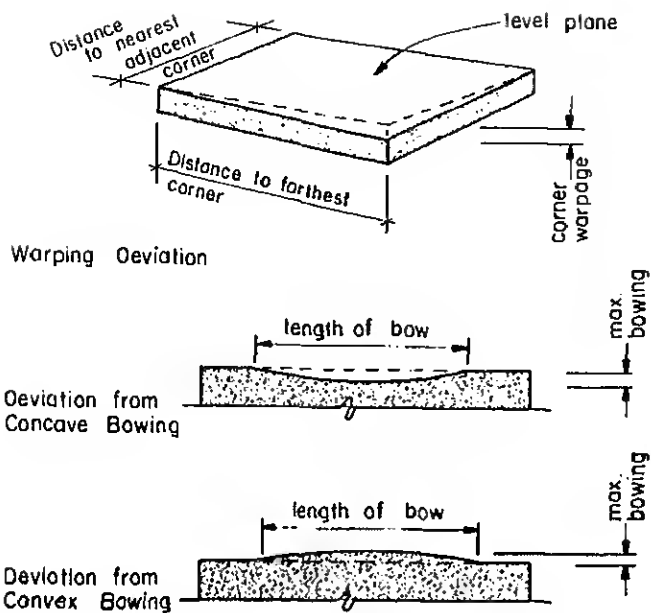


Fig. 40 Manufacturing Deviations

Errors in mold dimensions at the time of casting, errors due to incorrect filling of a mold, and the changes in the dimensions of the panel after compaction are considered to be the main sources of fabrication deviations for wall panels. (108,116-118) In floor elements, length error, caused by inaccuracies in cutting large panels into short pieces is the main manufacturing deviation.

#### 2.4.2 Setting Out Deviation

The measured space into which a panel is designed to fit is subject to setting-out deviations as shown in Fig. 39. These deviations are generally the result of limitations of measurement methods and equipment. The importance of setting-out deviations has been recognized and some guidance provided. (113,119-121)

out of line, it may be rotated in plan, or it may be out-of-plumb (Figs. 39,41). As panels are placed one after the other, their dimensional deviations can compound or cancel depending upon the direction of deviation. Assembly deviation can be predicted by use of "statistical theory of accumulated errors". (108,110,117,118,12

At present little information is available regarding the factors affecting assembly deviations, however, the weight and maneuverability of the panels seem to have some effect.

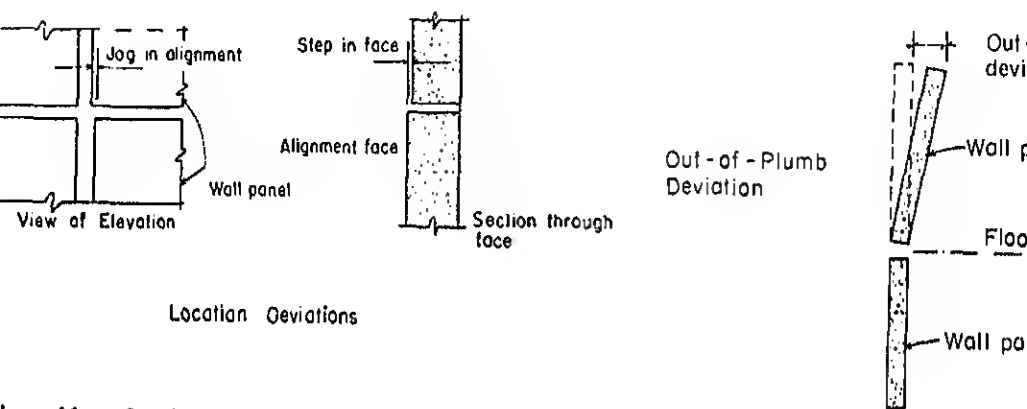


Fig. 41 Positional Deviations

#### 2.4.4 Table of Tolerances

A statistical solution for tolerances is not yet available. However, as a result of extensive experience, tolerances are becoming standardized in various countries through recommended practice and codes. (26,52,108,112,113) Based on a survey of several tolerance systems used in North America, the following tolerances seem to be reasonable:

Position of voids	$\pm 1/4$ in.
Position of tendons	$\pm 1/8$ in.
Position of weld plates	$\pm 1$ in.
Camber deviation from design camber after jacking	$\pm 1/8$ in. per 10 ft. not greater than $\pm 3$
Differential camber between members of the same design after setting and alignment	$1/4$ in. per 10 ft. b greater than $1/2$ in.
Squareness of ends - vertical and horizontal alignment	$\pm 1/4$ in.

### Dimensional Tolerances for Wall Panels

Length	$\pm 1/2$ in. up to 40 f
Height	$\pm 1/4$ in. up to 10 f
Thickness	$\pm 1/8$ in.
Sweep or Bowing	$3/8$ in. up to 40 f
Opening (Location/Size)	$\pm 1/2$ in.
Position of handling devices	$\pm 6$ in.
Position of weld plates or connection devices	$\pm 1/2$ in.
Diagonals	$\pm 1/2$ in. from plan

### Location Tolerances for Precast Panel Assembly

Face width of joints	$\pm 1/2$ in.
Joint taper	$1/40$ in. per ft. len $1/4$ in. maximum len ing in one directio
Step in face (Fig. 41)	$1/4$ in.
Jog in alignment of edge	$1/4$ in.



Due to production techniques, walls are generally more expensive than slabs on a per square foot basis. Hence, care is usually taken in layout and analysis to optimize the wall-to-slab ratio in LP structures.

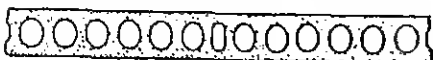
Typical wall elements in LP structures range in thickness from 6 to 12 in., in length from 10 to 45 ft., and in height from 8 to 10 ft. In general, wall panels have a section characteristic (i.e., solid, ribbed, cored, etc.), and a function characteristic (i.e., transverse or spine, interior or flank walls). The principal requirements for a load-bearing wall panel are adequate strength and deformability to satisfactorily accommodate both normal and abnormal loads. In design of wall panels the following considerations should be included:

- load and geometric eccentricities;
- slenderness effects;
- effects of connections;
- effects of openings;
- thermal effects; and
- minimum thickness.

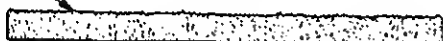
A detailed study of wall elements is presented in Report 3: "Wall Panels: Analysis and Design Criteria."

### 2.5.2 Effects of Clamping on Simply Supported Plank

Floor elements in LP residential structures in North America are mostly hollow core, precast, prestressed plank up to 8 ft. in width, of the cross-sections shown in Fig. 42. Structurally, f



Rough  
Surface



NOTE: This type is typically used only  
with structural topping

Fig. 42 Typical Cross Sections of Floor Plank

Floor planks are usually designed as simply-supported one-way slabs. The panels placed on the walls are free to deflect due to their self-weight. After the space between the top of the floor panels and the underside of the wall panel is drypacked, the end rotation of the floor panel is restricted by clamping action. This induces a negative bending moment over the support for loads subsequently applied and has the beneficial effect of reducing midspan deflection. However, when the bending moment exceeds plank sectional capacity, cracks may develop in the top of the plank near the wall faces (Fig. 43).

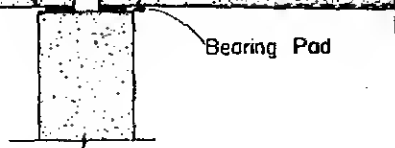


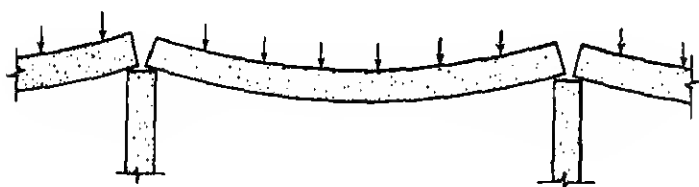
Fig. 43 Cracked Floor Panels at Supports

Recent unpublished tests<sup>(122)</sup> of slab panels on elastomeric bearing pads indicate that for superimposed loads typical of residential occupancies, the cracks (if any) at the supports are narrow and do not penetrate to the reinforcement level. No perceptible reduction in shear capacity of the plank was found. However, the results also suggest that where the ratio of superimposed load to dead load substantially exceeds one, the shear capacity is substantially reduced as the ultimate moment capacity is approached. In such cases to avoid dangerous shear failures, reinforcement should be provided at the slab ends.

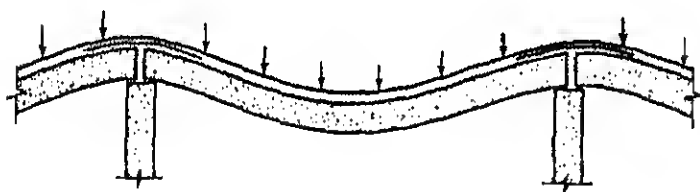
### 5.3 Composite Plank, Moment Continuity and Horizontal Shear

Heavy loadings or fire considerations require greater flexural capacity than that of the most heavily prestressed section of a given thickness. In such cases negative moment continuity at the support can help to reduce the maximum positive bending moment. Tests,<sup>(122)</sup> however, have indicated that continuity reinforcement placed in the grouted key joints and clamping action of wall panels provide only limited negative moment continuity.

... and loads, and partition loads (Fig. 44b). Loads due to the self-weight of the plank, which usually constitute at least 50% of the total load, are carried by simply supported spans (Fig. 44a), unless the planks are shored while continuity is established.



(a) Simply Supported Member



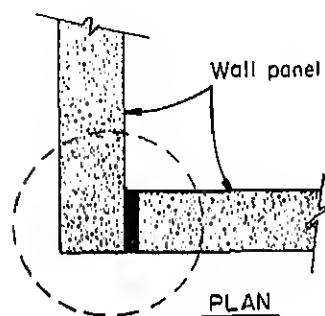
(b) Continuous Member

Fig. 44 Analysis Techniques for Floor Plank

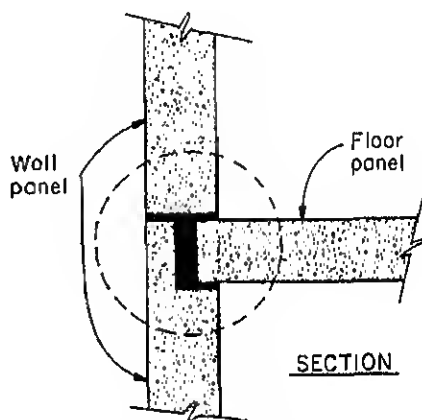
Horizontal shear capacity at the plank-topping interface should be assured by increased interface bond. Inadequate horizontal shear capacity at the interface due to excessive shrinkage of the topping can cause the topping to act independently, thus impairing the beneficial effect of composite action between the topping and the plank.

where the edges of shafts are chamfered more than 2 in. the bearing length must be measured from the inside edge of the chamfer. Bearing lengths can be reduced from the values noted above if the bearing surfaces are armoured. However, short bearing length imposes tight and potentially costly manufacturing and erection tolerances. (123)

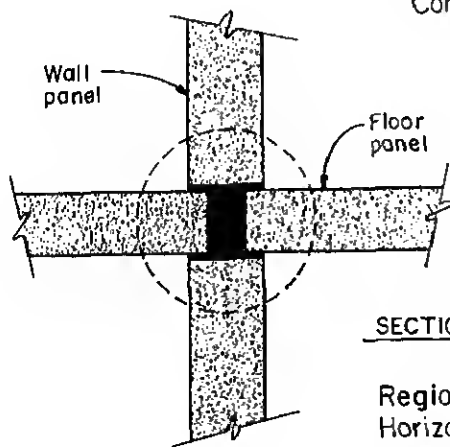
precast elements embedded in the joint. The function of connection is to transfer forces from one element to another. If sufficient strength and ductility is not developed in the connections, the available strength in the adjoining elements may not be fully utilized. As a result, the design and detailing of connections and continuous joints is an important factor in determining the safety and economic feasibility of precast structures. (124,125)



Region of Vertical  
Wall-to-Wall Connection



Region of Exterior  
Horizontal Wall-to-Floor  
Connection



Region of Interior  
Horizontal Wall-to-Floor  
Connection

- deflection, etc.;
- (c) practical problems of casting and erection, including  
ances, etc.; and
- (d) inaccessibility of joints which prevents inspection and  
rectification.

Most procedures developed for connection design in panel structures are of European origin, with European connections in mind. Therefore, it is necessary to formulate complete and rational procedures and design criteria for connections in American LP structures to include all factors affecting strength and behavior under normal and abnormal loading conditions. In the following text, strength and performance requirements for the most widely used connection details are discussed to provide a basis for analysis and design. A detailed discussion of specific analysis and design methods, and suggested acceptance criteria will be given in a subsequent report.

Connections are classified with respect to location, direction, and function, e.g., interior or peripheral, horizontal or vertical, wall-to-wall or wall-to-floor. The primary connections in an LP structure are divided into the following distinct groups (Fig. 46):

1. Interior horizontal wall-to-floor;
2. Exterior horizontal wall-to-floor;
3. Horizontal floor-to-floor; and
4. Vertical wall-to-wall.

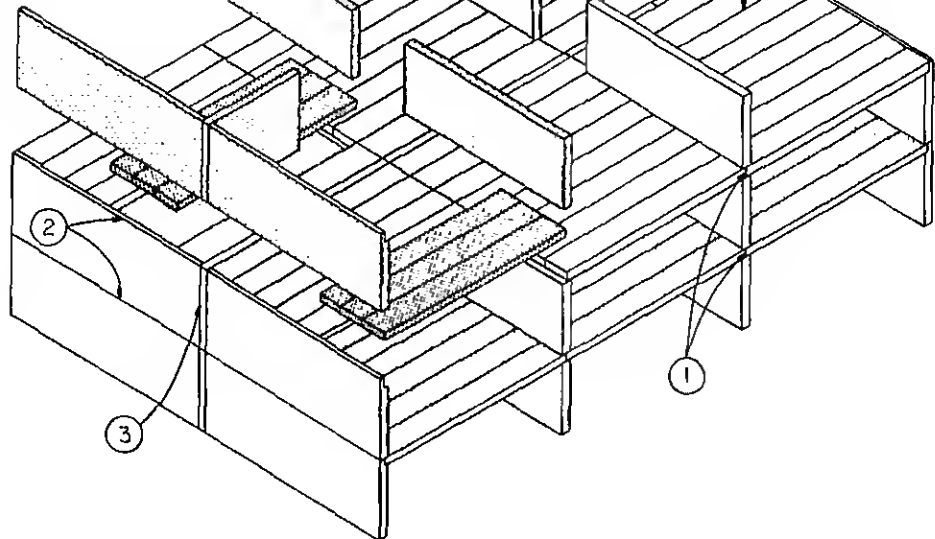


Fig. 46 Connection Classification and Location

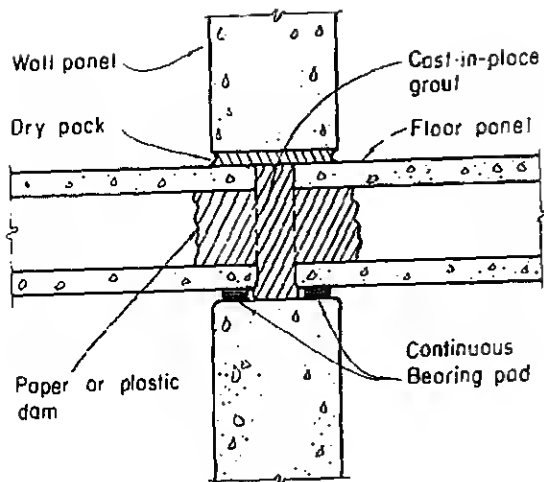
are also secondary joints, e.g., between lintels and wall panels, however, since their design is usually straightforward, they will be discussed in this report.

### 2.6.1 Interior Wall-to-Floor Connections

This connection is also termed a horizontal function, a horizontal joint or a compression transfer connection. Various details are currently in use for this connection.<sup>(51-53,126-134)</sup> Most widely used among these is the "platform" connection (also known as the "closed" or "American" type connection) shown in Fig. 47.

In the typical "American" connection, prestressed hollow core slab extending over wall panels are continuously supported on bearing pads. Cast-in-place grout fills the vertical space between floor plank and usually a portion of the hollow cores of the plank itself. In the latter case paper or plastic dams are inserted into





( Ties not shown )

Fig. 47 Typical Interior Wall-to-Floor Connection

The function of the interior horizontal connection is to successfully transmit the forces described below and illustrated in Fig. 48:

$V_1$  = vertical load from wall panel above;

$V_2$  = vertical load from floor loading;

$H_1$  = horizontal shear due to lateral loads on wall;  
this force also is present under abnormal

$H_3, H_4$  = horizontal thrust in the plane of the floor (tension or compression) resulting from diaphragm action and restraint force due to creep, shrinkage and temperature effects; and

$M$  = transverse bending moment from floor continuity and load eccentricities.

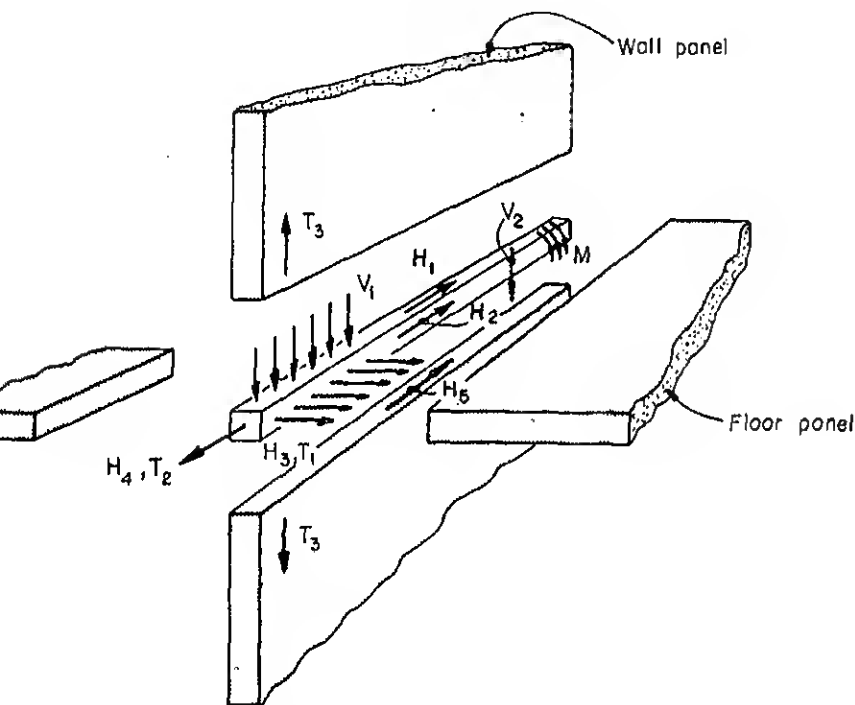


Fig. 48 Exploded View of an Interior Floor-to-Wall Connection and Various Connection Forces

to fulfill the requirements of General Structural Integrity described in previous sections, the connection must also satisfactorily resist the following forces and moments:

action in the wall;

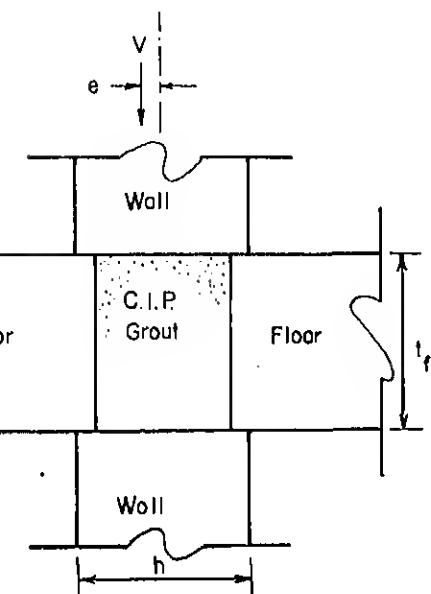
- $H_1$  = horizontal beam shear force between the edges of wall panels and the joint as a result of integral cantilever or beam action (see Figs. 24 and 27); also present under normal loads;
- $H_5$  = horizontal shearing forces to transfer the tie force  $T_2$  into the panel below; and
- $T_3$  = vertical tie forces to provide suspension action and dowel action for developing  $H_1$  and  $H_5$ .

#### 2.6.1.1 Variables Affecting Connection Behavior

The connection is composed of various parts with forces acting concurrently. Consequently the state of stress in the connection is quite complex. In the last decade many significant studies of the behavior of connections have been reported.<sup>(104,122,125-127,129-138)</sup> The following is a summary of the results relevant to the "American" type connections.

Precast and Cast-in-Place Concrete Properties--Variations in the elastic moduli of the wall concrete, floor concrete, and the cast-in-place concrete appreciably affect the behavior of the connections.<sup>(91)</sup> Their relative values influence stress distribution within the connections and consequently the load-carrying capacity. Lugez<sup>(132)</sup> observed that separation of the floor panels from cast-in-place grout in the connections occurs under vertical compressive loading of very low magnitude. Thus, continuity between the floor panels and the

study concluded that tensile strain increases with reducing values of the elastic modulus of the cast-in-place grout,  $E_j$ .



$e$  = eccentricity of load (varies)

$t_f$  = floor thickness = 7.1 in. (180 mm)

$\alpha$  = penetration of floor panel into joint (varies)

$h$  = wall thickness = 7.1" (180 mm)

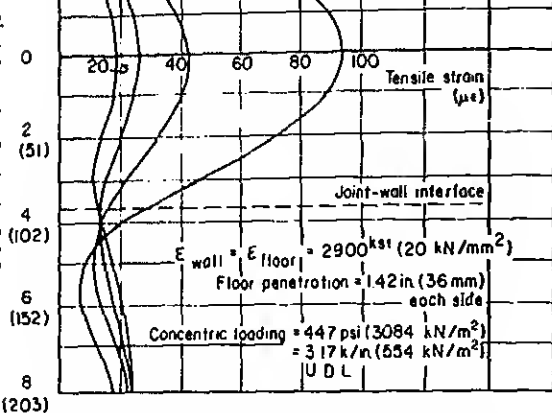
$V$  = vertical load

## 9 Platform Connection Details used in Ref. 91

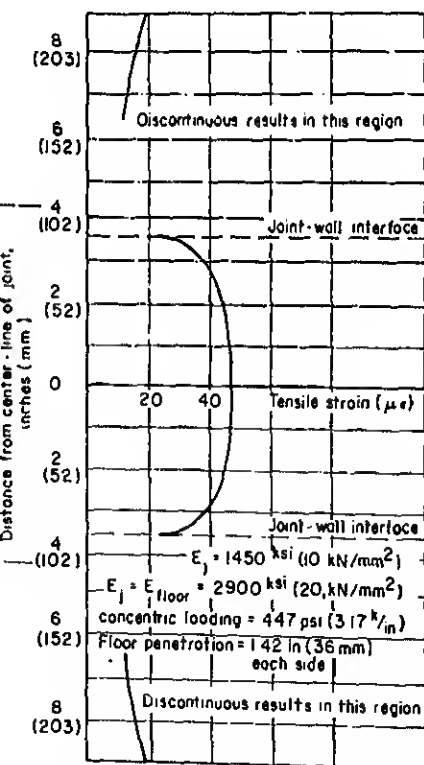
Depending on the values of the elastic moduli of the wall, floor and cast-in-place grout, a horizontal connection under concentric loading can fail in four basic modes, as shown in Fig. 52.

The connection fails as shown in Fig. 52a when concrete in the precast wall panel develops compression cracks near its end. This mode predominates if wall concrete is weaker than floor and cast-in-place grout.

When the hollow cores in the floor plank are left unfilled at the support, or when the floor plank is supported on "softer" material such as elastomeric or other bearing pads, the

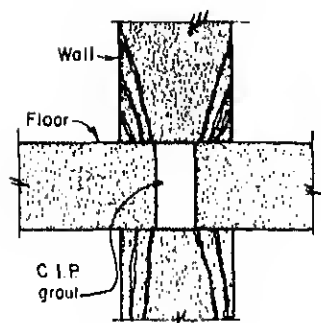


50 Tensile Strains along the Centerline in the Connection shown in Fig. 49 (Ref. 91)

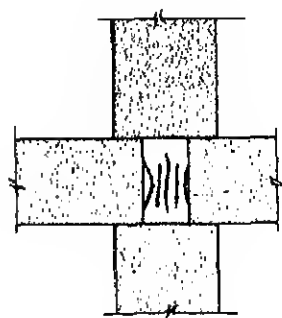


51 Tensile Strains along the Centerline of a Cracked Connection Shown in Fig. 49 (Ref. 91)

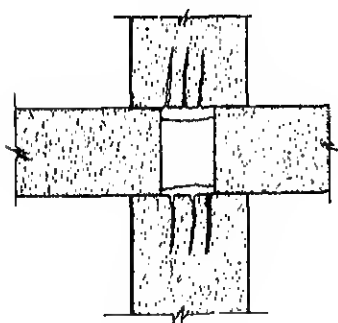
the floor plank is not eliminated by tensile forces, the is in a biaxial state of stress and can fail in compression as illustrated in Fig. 52b.



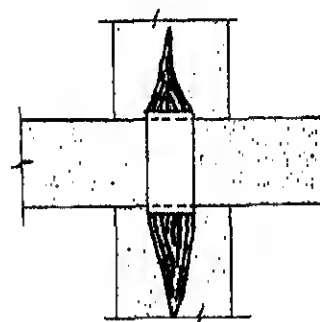
(a) Failure of Wall under Compression



(b) Failure of Cast Grout



(c) Weak Cast-in-Place Grout and Strong Floor Concrete Cause Splitting of Wall Panel



(d) Failure through Grout of Strong Cast-in-Place Grout into Wall Panel

Fig. 52 Various Failure Modes of an Interior Wall-to-Floor Connection

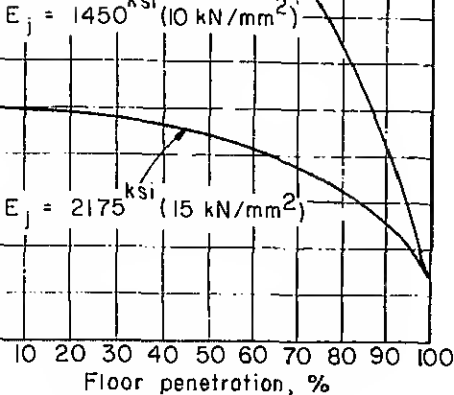
In the case where the cast-in-place grout is stiffer than the wall panel and the floor plank, the cast-in-place grout supports almost the entire load from the wall above. Under these conditions, the cast-in-place grout wedges into the wall panel causing the splitting shown in Fig. 52d.

Experimental studies have shown that splitting of wall panels is of major concern.<sup>(52,91,134)</sup> As a result capacity reduction factors for reinforced concrete walls have been suggested. Alternatively, test results have shown that splitting can be minimized by use of proper transverse split-resisting reinforcement at wall panel ends.<sup>(134)</sup> Based on experimental tests related to American practice, definitive recommendations will be presented in a subsequent report on connection design.

Effects of Floor Penetration--Floor penetration is defined as the measure of floor end-length bearing on the wall. The ratio of penetration per unit wall thickness is termed the penetration ratio,  $p$ , i.e.:

$$p = \frac{\text{sum of two floor panel end lengths bearing on wall}}{\text{wall thickness}}$$

Theoretical and experimental studies of solid slab sections have been carried out in England indicate that as the penetration ratio increases the tensile strain in the wall decreases.<sup>(91)</sup> For the connection configuration shown in Fig. 49, this strain increases as  $E_j$  of the cast-in-place grout decreases (Fig. 53).



Concentric loading = 447 psi

(3084 kN/m<sup>2</sup>)

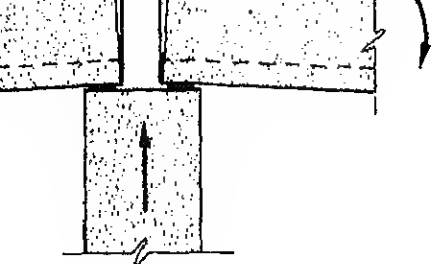
$E_{\text{wall}} = E_{\text{floor}} = 2900 \text{ ksi} (20 \text{ kN/mm}^2)$

8 Effect of Floor Penetration on the Tensile Strains at the Center of the Connection in Fig. 49 (Ref. 91).

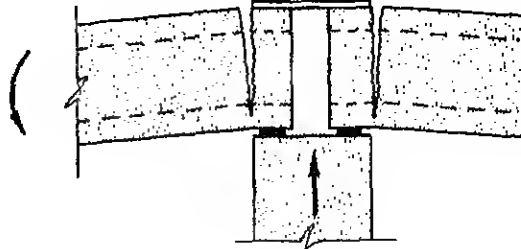
Floor Moment--Floor moment can be introduced in the connection if rotation of the floor panel at the joint is restricted by vertical clamping from the wall panel. Depending on the magnitude of clamping force and on the negative moment capacity of the plank, either of two modes of cracking exist:<sup>(122)</sup>

1. Separation at the interface between plank and cast-in-place grout in the connection can occur when the clamping force is insufficient to prevent plank rotation or when measures were taken to allow such rotation (Fig. 54a). Such rotation induces splitting forces in the wall above.
2. Alternatively, cracking in the plank near the support can occur when the induced moment exceeds the plank's moment capacity (Fig. 54b). This condition





rotates due to  
insufficient clamping  
at joint



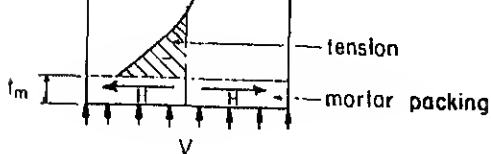
(b) Slab cracks due to  
insufficient flexural  
capacity

Cracking Modes at Interior Wall-to-Floor Connection due to  
Slab Moment

Load Eccentricities--The load carrying capacity of a connection can be reduced under load eccentricities. For a given eccentricity, the reduction in connection capacity depends upon various parameters: wall thickness, penetration ratio, and properties of cast-in-place and precast concretes. (132)

Design charts are available to determine the reduction in capacity due to load eccentricities. (91,132)

Effect of Mortar Packing--After a wall is vertically adjusted, the space between the bottom of the wall and the floor slabs is filled with a drypack mortar. Workability, proper compaction and shape of drypack influence the capacity of the connection. Incomplete grouting or packing reduces the capacity of the connection substantially and greatly effects its overall behavior. If the drypack is poorly compacted at the center of



$h$  = Wall thickness  
 $t_m$  = Mortar thickness  
 $H$  = Horizontal force  
 $V$  = Vertical force

Fig. 55 Theoretical Transverse Stress Distribution on a Wall (Ref. 132)

the connection, loads which would normally pass through the center grout will be transferred through the plank instead. This condition can cause premature splitting of the wall as shown in Fig. 52d.

If the stress-strain characteristics of the packing material is significantly different from that of the wall, the lateral deformation of the mortar can generate horizontal forces at the bottom of the wall (Fig. 55). Finite element analysis has shown that if the mortar is of poor quality, or relatively thick, the magnitude of tensile stresses becomes sufficiently high to cause cracking and splitting of the wall end.<sup>(132)</sup> European experimental and theoretical studies indicate that the problem can be minimized if the following steps are taken:<sup>(52,132)</sup>

- the connection is properly made with well compacted mortar;
- the strength of the mortar is equal to or not less than 75% of the concrete strength of the wall; and
- the mortar thickness,  $t_m$ , is not greater than

considerable horizontal shear from the wall above to the panel below under both normal and abnormal loading conditions. If shear transfer mechanisms are not available, brittle failures can occur, particularly under conditions presenting the damaged state.

When the connection is under compression, a portion of the shear can be transferred by friction between the precast concrete wall panels and the connection. The magnitude of this shear force,  $S$ , is determined by the relationship:

$$S = \mu N \quad (\text{Eq.})$$

where

$N$  = compressive force normal to the direction of interface; and

$\mu$  = coefficient of friction between wall panels and mortar layer.\*

If the magnitude of induced shear force exceeds the force resisted by the vertical reinforcing, additional vertical reinforcing is necessary. The required vertical reinforcing steel area,  $A_{vf}$ , can be conservatively calculated by using the shear friction analogy equation:<sup>(125)</sup>

$$A_{vf} = \frac{H}{f_y \tan \alpha} \quad (\text{Eq.})$$

---

\*In various specifications the value,  $\mu$ , varies from 0.2 to 0.8. (

$\tan \alpha =$  coefficient of internal friction between precast wall and the drypack or grout, normally taken as 0.7. (59,104,139)

The simple shear-friction approach to designing joints has been adopted by the ACI code.<sup>(15)</sup> It assumes that the reinforcement across the crack acts as a tension tie rather than as a dowel. The cohesive strength of the concrete as well as the effect of compressive force across the connection are neglected. Since the interface inherently provides a convenient place for slippage to occur, the assumption to neglect the cohesive strength seems justified.

## 2.6.2 Exterior Wall-to-Floor Connection

This connection, which typically occurs only at bearing flank walls of the structure (Fig. 46), is also termed an exterior horizontal connection. Various details are currently in use.<sup>(52-4,102, 133,134)</sup> As in the case of the interior horizontal connection, the "American" or "platform" type joint seems to be the most widely used (Fig. 56). In this detail the hollow core floor plank extends over the wall panels from one side only supported on bearing pads. The space between the plank and wall is filled with cast-in-place grout with paper or plastic dams inserted into the hollow cores to limit the extent of core filling. The void between the top of the plank and the bottom face of the wall panel is filled with drypack mortar while the exterior face of the junction between the wall panels is caulked.

The forces and actions which exist in the exterior horizontal

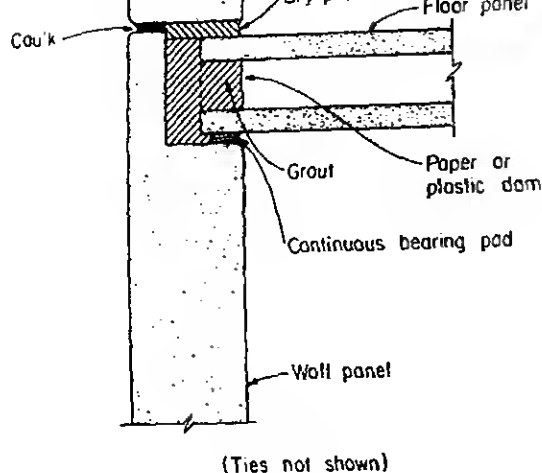
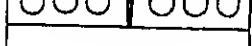


Fig. 56 Typical Exterior Wall-to-Floor Connection

connection. The variables affecting its behavior are also similar to those discussed for the interior horizontal connection. However, the lack of symmetry in the exterior connection causes an eccentric stress distribution which reduces its load-bearing capacity. Tests indicate that the ultimate stress for this connection can be as low as  $0.25f'_c$ , as compared to  $0.85f'_c$  for a concentrically loaded wall panel.<sup>(140,142)</sup> In addition, the eccentricities induced at the connection significantly affect the behavior and design of the exterior walls.

### 2.6.3 Longitudinal Connection between Floor Elements

This connection, which occurs at adjacent vertical faces of individual floor elements, is usually termed a "shear key". As indicated in Figs. 57 and 58, various profile and reinforcement details are used for this connection.




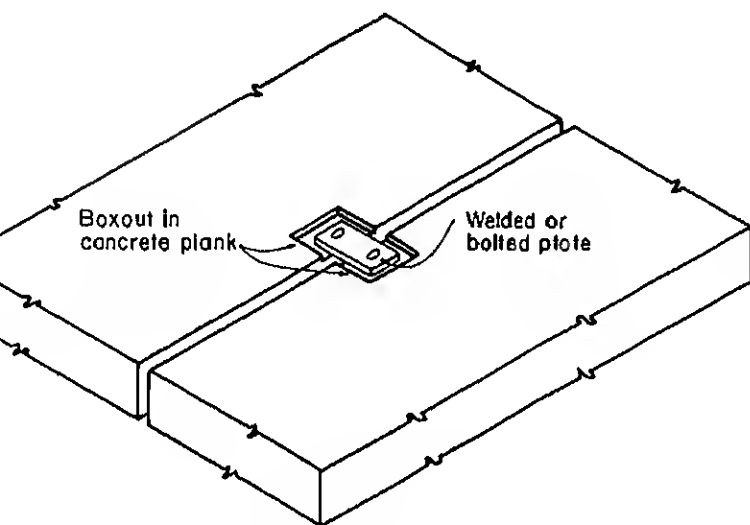
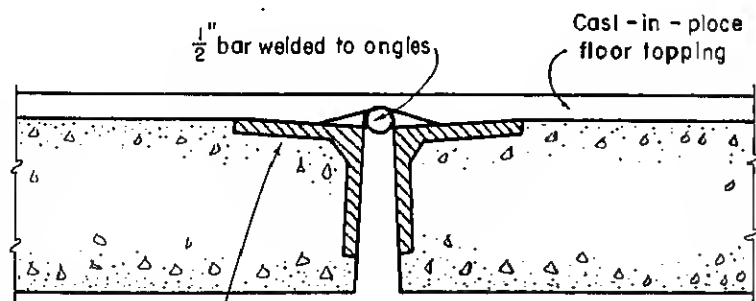
 indicates Cast - In - Place Grout

Fig. 57 Grouted Connections Between Floor Elements



Bolted Plate



Short lengths of  
angle cast into  
floor units

Welded Bar

. 58 Mechanical Connectors Between Floor Elements

The joints introduce discontinuities which can adversely affect the floor's overall performance. To minimize such effects, the horizontal connection or shear key must resist with minimal deformation the following (Fig. 59):

$V_3$  = vertical (out-of-plane) shear induced from adjoining floor/roof elements subject to varying superimposed loads:

$H_6$  = horizontal (in-plane) shear resulting from the floor's diaphragm action; and

$H_7$  = normal force (tensile or compressive) resulting from the floor's diaphragm action.

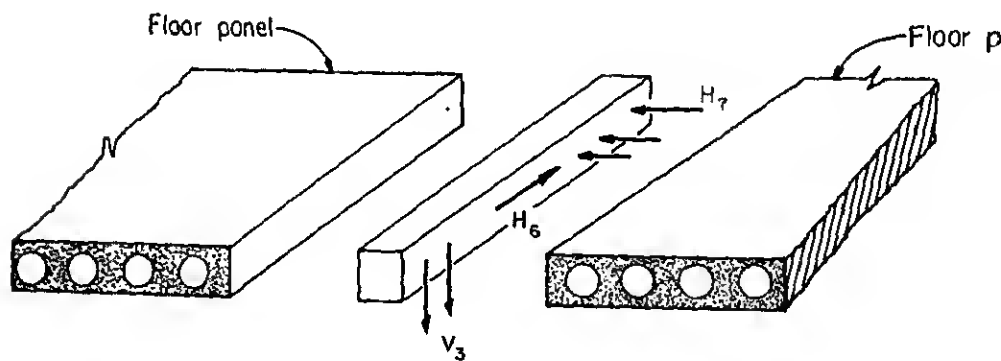


Fig. 59 Exploded View of the Horizontal Connection Between Floor Panels and the Various Connection Forces

European type precast panels can act as effectively as a monolithic slab in horizontally stiffening a structure. The study determined the effects of longitudinal connections on the in-plane deformation of the following types of floors:

- (a) solid cast-in-place;
- (b) precast panels with grouted connections enclosed with a reinforced concrete ring beam; and
- (c) precast panels with connections left ungrouted, enclosed with a reinforced concrete ring beam.

The load-deformation relationships of these floor systems are shown in Fig. 60. The force-deflection curves illustrate that the ungrouted connections appreciably reduce the overall floor rigidity. However, the floor with grouted connections acts as effectively as a monolithic slab, until tensile cracking in the ring beam occurs.

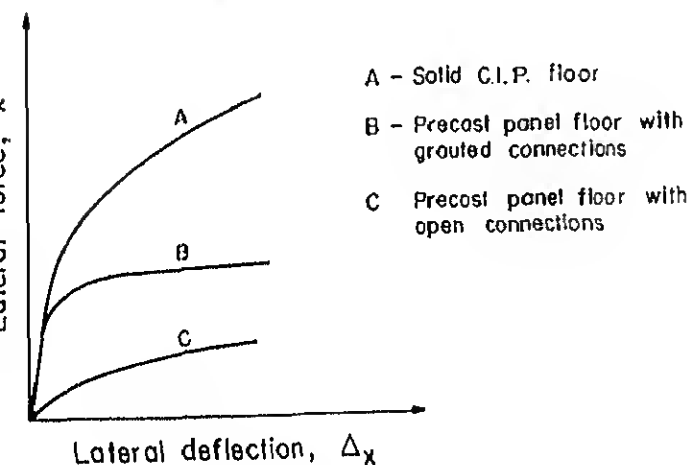


Fig. 60 Force-Deformation Relationships of European Types of Floor Systems under Horizontal Loading



the keys is approximately  $3\sqrt{f'_c}$ .<sup>(143)</sup> For typical configurations shear stresses developed in the horizontal connection between adjacent plank should not exceed this value.

Based on the above discussion, two conditions should be fulfilled to achieve diaphragm action:

1. The keyways between precast panels should be filled with grout or mechanically fastened together, unless an adequate topping slab is employed in the design; and
2. Peripheral reinforcement should be adequate to prevent the formation of tensile cracks under service load conditions.

#### 2.6.3.2 Behavior under Vertical Shear

Vertical shear is induced in the keyway when adjacent panels are subjected to different vertical loads, e.g., one panel supports a concentrated load, such as a partition wall, while the adjacent panels carry only their own weight. The purpose is essentially to deflect the adjoining panel edges, ensuring participation of such panels in the structural response.

Tests on slabs with grouted shear keys<sup>(143)</sup> show that the ultimate vertical shear stress of the keys is approximately  $3\sqrt{f'_c}$ . Although shrinkage of the grout in the key can cause a crack of 0.001 in. (.03 mm), such a crack width is

#### 2.6.4 Vertical Wall-to-Wall Connection

This connection occurs at the vertical edges of adjoining wall panels within interior or exterior wall assemblies. The various details used for this connection are classified as dry or wet according to the methods and devices employed.

The vertical connections used in American practice are almost exclusively of the "dry" type, which increase the speed of erection. They consist mostly of steel plates or angles welded or bolted to inserts cast within the panel (Fig. 61).

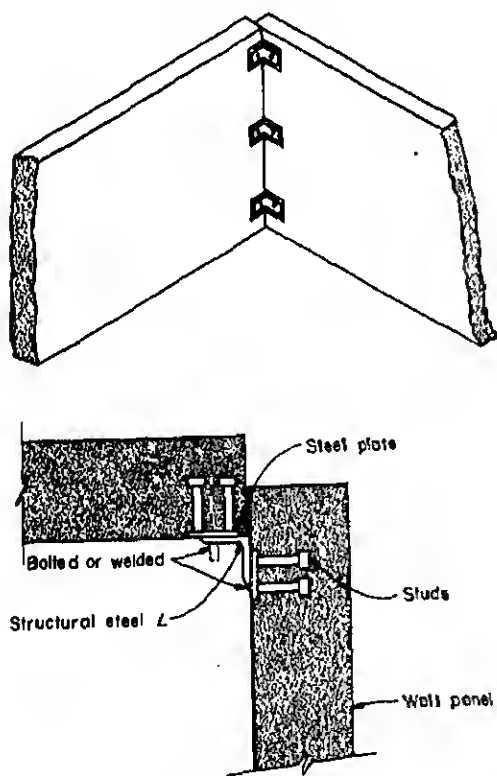


Fig. 61 Typical "Dry" Vertical Wall-to-Wall Connection

reinforcing links are cast into the edge of the panel and project into the connection grout. To optimize the use of "special" connections the reinforcing loops protruding from the panels are interlaced by one or more vertical reinforcing links before the grout is placed.

Under normal loadings the basic function of a vertical connection is to equalize deformations between adjoining stack panels through in-plane shear from one stack to the next. Under abnormal loading conditions deformations can result from:

- (a) different stress levels in the panels;
- (b) different moduli of elasticity of the panels;
- (c) varying creep, shrinkage and temperature effects in the panels.

When the vertical connection is located within a wall panel subjected to lateral forces, shear ( $V_d$ ), and compressive stresses ( $H_g$ ) will be transferred through the connection. This is shown in Fig. 63.

Abnormal loading conditions significantly increase the shear requirements for the connection. This is due to lever or beam action that is developed in the wall panel. If the lower panel becomes ineffective, under these conditions the vertical connection must act as a support for the upper panel on the panel. As a result, the magnitude of shear required for the connection is specified for abnormal conditions will usually govern the design of the connection for shear.

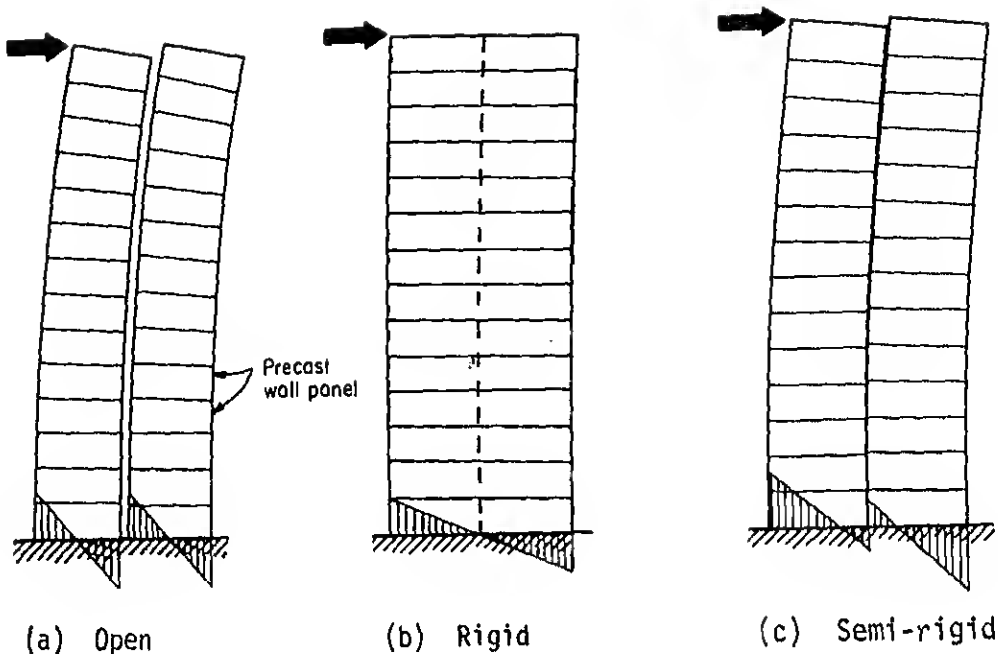


Fig. 64 Drift and Stress Distribution in Shear Wall Assemblies

Open Connection--With this type of connection no shear force is transmitted from one stack of panels to another. As a result, the vertical edges are free to slide past each other (Fig. 64a). In practice the open connection should be avoided since (a) it can cause slab cracking at the connection, and (b) slip at the vertical interface significantly reduces overall stiffness.

Rigid Connection--A vertical connection is called rigid when there are no relative displacements of the adjoining wall panels. A rigid connection provides full interaction between

Semi-rigid Connection--In practice some slip occurs in almost all vertical connections. (145,146) The shear rigidity of the semi-rigid connection is less than the rigidity of the stacks of adjoining wall panels. As a result, the shear stress transferred across the connection is accompanied by a degree of slip between the adjoining wall panels (Fig. 64c).

For dry joints the rigidity primarily depends on connector strength and type. The various connection types (i.e., bolted and welded), have different force-deformation characteristics that influence the degree of slip in the connection.

For wet connections the rigidity can be characterized by a rigidity modulus,  $G_j$ , defined as the ratio of an increment of shear stress divided by a corresponding increment of shear displacement. Factors affecting the rigidity of a wet connection include:

- (a) material properties of cast-in-place grout and reinforcing steel;
- (b) amount and distribution of reinforcing steel;
- (c) connection width;
- (d) shape and spacing of vertical keys (castellation);  
and
- (e) height of the structural wall.

Slip can be minimized in either connection with proper attention given to the details. For purposes of analysis a nominal amount of slip is insignificant.

system of forces and moments, the connection can fail in one of the following modes:

- (a) metal connector failure;
- (b) insert pull-out failure; or
- (c) concrete failure.

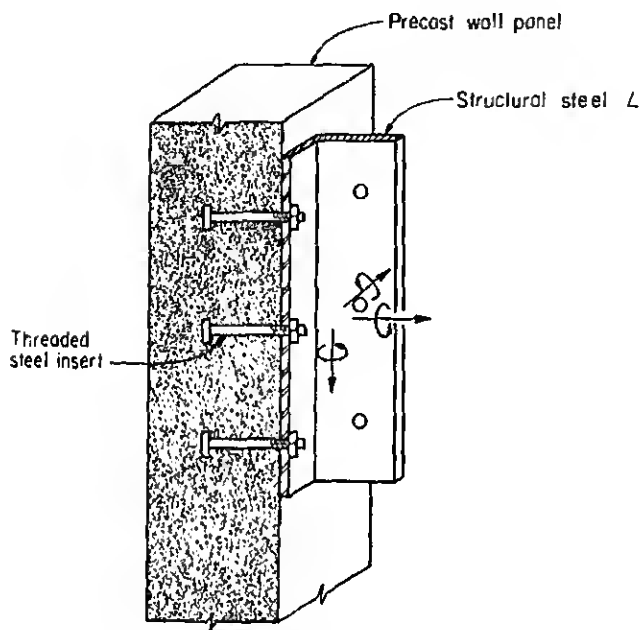


Fig. 65 Forces and Moments on a Dry Vertical Connection

Failure mode (a) occurs when the steel angle or insert yields. Information on the nature of such failure, as well as design procedure to prevent this failure mode, is readily available. (147-149) In mode (b), the entire insert can pull out of the concrete due to bond failure between concrete and

tension failure occurs in the form of a concrete cone having its apex slightly past the deepest part of the insert, as shown in Fig. 66. Diagonal tension failure takes place when the tensile strength of the "shear cone" surrounding the insert is less than the strength of the anchoring elements of the insert.<sup>(152)</sup> The results reported in Ref. 152 indicate that the embedment length of an insert is the most important factor in determining the pull-out capacity of the insert. Various design formulae to determine ultimate pull-out strength of studs and inserts are available.<sup>(113,152)</sup>

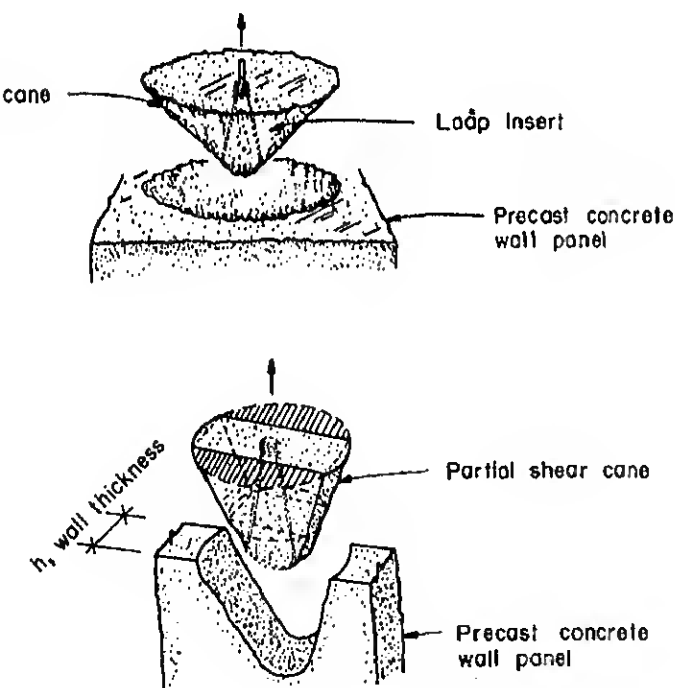


Fig. 66 Concrete Cone Failure at Ultimate Pullout Load

elastic modulus of the concrete. If adequate concrete bearing area or strength is not available, bearing failure can occur. Empirical formulae to calculate the allowable shear capacity of a stud bolt are readily available. (113)

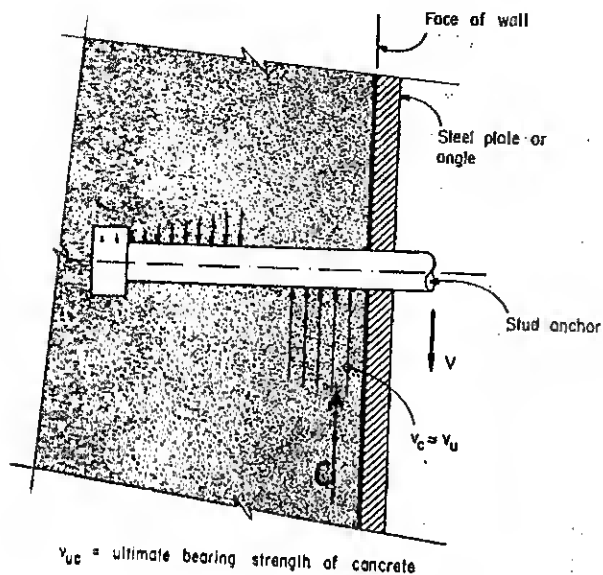


Fig. 67 Stress Distribution at Concrete Bearing Failure of Dry Vertical Connection

#### 2.6.4.3 Behavior of Wet Vertical Connections

Many notable European studies have been performed on the wet connection, and considerable material is available. (91,124,133,135,137,145,146,153-155) A brief summary is presented below.



above 85 psi. Such connections are subject to a sudden brittle failure. Plane reinforced connections, on the other hand, are relatively ductile, and can develop considerable ultimate shear resistance<sup>(124)</sup> through dowel action and shear friction.

Grooved--Unreinforced grooved connections can exhibit a very high rigidity because the groove increases the bond surface. However, as in the case of unreinforced plane connections, such resistance is not dependable, due to loss of bond resulting from shrinkage. Reinforced grooved connections exhibit higher elastic resistance than reinforced plane connections, while their ultimate shear resistance is approximately the same.<sup>(53,91,124)</sup>

Keyed--Keyed connections are the most commonly used wet vertical connections. The elastic resistance against slip is provided by interlocking rather than by bond. Once the principal tensile stress in the connections exceeds the tensile resistance of the cast-in-place grout, cracks develop and shear resistance decreases. In a reinforced section after cracking, the shear force is transferred across the connection through diagonal strut action, with the compressive and tensile components of the strut action resisted by the concrete and steel respectively.<sup>(145)</sup> Empirical formulae to determine maximum shear resistance of keyed connections are available. (

tance to structural safety. Recognizing this, and considering the many uncertainties in connection design, various codes have recommended that connections be designed with higher safety factors than the adjoining elements. The higher safety factors may be further justified in light of the connection's susceptibility to inferior quality control during construction.

$A_{vf}$  = area of vertical reinforcement

$A_w$  = cross-sectional area of the wall

$b$  = length of a precast wall panel

$C$  = compressive force on concrete

$C_u$  = ultimate load-carrying capacity of a connection in compression

$e$  = eccentricity of compressive loading on a wall from centroidal plane of the wall

$E_{floor}$  = elastic modulus of concrete in floor panels

$E_j$  = elastic modulus of cast-in-place grout in connection

$E_{wall}$  = elastic modulus of concrete in wall panel

$f'_c$  = compressive strength of concrete

$f'_{cf}$  = compressive strength of concrete in precast floor panels

$f'_{cj}$  = compressive strength of cast-in-place grout in connection

$f'_{cw}$  = compressive strength of concrete in precast wall panels

$f'_m$  = compressive strength of mortar

$f_s$  = stress in steel

$f_y$  = yield strength of reinforcing steel

$G_j$  = rigidity modulus (shear modulus) defined as the ratio of an increment of shear stress divided by a corresponding increment of shear displacement

GSI = general structural integrity

$h$  = precast wall panel thickness

$H$  = tangential shear along an interface

$h$  = height of precast wall panel

- $p$  = penetration ratio = 
$$\frac{\text{sum of two floor panel end lengths bearing on wall}}{\text{wall thickness}}$$
- $p$  = ultimate pull-out capacity of stud or insert as given by concrete
- $S$  = shear force
- $SF$  = safety factor of design
- $t_f$  = thickness of floor panel
- $U$  = required strength to resist design loads
- $v_{uc}$  = ultimate bearing strength of concrete
- $\alpha$  = angle of internal friction between precast panel and mortar layer
- $\mu$  = coefficient of friction between wall panel and mortar layer
- $\gamma$  = load factor

	codes and design practices.
Acceptable risk:	That level of risk deemed tolerable.
Accidental eccentricity:	An eccentricity which exists as a direct result of errors in either manufacturing or erection procedure.
Actual risk:	See "risk".
Actual size:	A size found by measurement.
Alternate path:	The substitute load flow pattern taken after an abnormal loading caused the "ineffectiveness" of a primary load-bearing element.
Assembly:	An aggregate of panels.
Bearing capacity:	The maximum unit pressure which a material will withstand without failure or without settlement to an amount detrimental to the integrity or the function of the structure.
Brittle connection:	A connection which depends on friction and bond under compressive loadings and consequently exhibits little or no ductile characteristics.

clearance:	Distance considered in the selection of a work size in order to achieve
connection:	A position or region where two or more building components, panels or assemblies are put together or united.
continuity:	The capacity for load transfer between two or more elements where load is axial, shear, moment, or a combination thereof.
coordinating dimension:	Dimension of the spatial envelope containing the precast panel.
cross-wall system:	A large panel system in which the load-bearing walls are perpendicular to the longitudinal axis of the building.
deflection index:	Ratio of the lateral deflection to the building height.
deformation:	A change in dimension or shape due to stress.
deviation:	The difference between a size or position (actual, limit, etc.) and specified size or position.

confined area; also, the mixture is  
placed.

packed concrete:

A concrete mixture sufficiently dry  
to be consolidated by heavy ramming.

ility:

The measure of a structural component's (element or connection) ability to sustain inelastic deformations, i.e., the ratio of the maximum deformation to the yield deformation.

ent vulnerability:

The susceptibility of a structural element to the effects of an abnormal load due to its location within the structure.

ure:

A loading or deformation state beyond which a connection, member or structure ceases to fulfill its function.

ck wall assembly:

An assembly of wall panels along the periphery of the building.

or panel:

A horizontal precast concrete element reinforced with mild or high-strength steel.

or plank:

A horizontal precast concrete element typically extruded and reinforced with high-strength steel. Also known as

to inhibit progressive collapse while retaining structural stability; achieved through a degree of continuity combined with a degree of ductility of the components and connections of a structure.

ut:

Mixture of cementitious material and aggregate to which sufficient water is added to produce pouring consistency without segregation of the constituents.

uting:

The process of filling with grout.

duced deviation:

Deviation induced by human error and the limits imposed by measuring instruments.

ffective element:

An element which has been adversely affected by an abnormal load so that it no longer functions as a load-bearing member to the extent originally designed.

erent deviations:

Deviations in shape and position of panel in service.

egral column:

A column located within a wall panel which can function as a vertical support under abnormal conditions.



such units.

panel structure:

A structural system composed of vertical load-carrying elements of large precast wall panels with precast floors and roofs of panels or planks.

-bearing wall:

A wall designed and built to carry superimposed vertical and shear loads, as opposed to non-load-bearing walls.

factor:

The ratio of the ultimate collapse load to the working load in a structural section.

flow:

The path taken by a load through the structural assembly from its point of application to its point of resistance (foundation).

longitudinal tie:

The horizontal tie used to link adjacent spans of floor elements.

mechanical bond:

The bond attributed to keying or interlocking rather than adhesion.

cross-wall system:

A large panel system composed of a combination of cross-wall and spine wall systems.

Optional removal:	The approach used to establish integrity within a structure in which element or portion of the structure is assumed totally removed.
Partial stability:	Stability of the structure in the damaged state.
Penetration ratio:	Defined as the ratio of the sum of two floor panel bearing end lengths to the wall thickness; used for interior horizontal connections.
Peripheral tie:	The horizontal tie used to create ring beam at the exterior of each story.
Progressive collapse:	The phenomenon in which the spread of an initial local failure eventually results in the collapse of an entire building or a disproportionately large part of it.
Recipient panel:	A floor panel which is subject to debris loading.
Return wall:	A vertical element used as a continuation of a load-bearing wall element at a right angle to it.

pecified dimension:

Dimension required for panel fabrication so that the panel, when assembled, will comply with spatial discipline.

pine-wall system:

A large panel system in which the load-bearing walls are parallel to the longitudinal axis of the building.

structural eccentricity:

An eccentricity which exists as a direct result of a structural defect.

tolerance:

The allowable deviation from a specified dimension.

traditional loading condition:

See "Normal loading condition".

Transverse tie:

The horizontal tie used to laterally link adjacent wall elements in the same vertical plane.

Ultimate load:

The maximum load which may be placed on a connection, member or a structure before its failure; also, the load at which a connection unit or structure fails.

Vertical tie:

The tie used to vertically link adjacent lifts of wall elements in the same plane.

either load-bearing or non-load-bearing; usually one-story in height with lengths typically ranging from 10' to 45'.

Warping:

A deviation of a floor or wall plane surface from its original shape, usually caused by temperature or moisture differentials within the slab or wall.

...solve the problems associated with abnormal loadings. However, the primary initiative and responsibility for defining, quantifying, and guiding belongs to those regulatory agencies charged with drafting codes of practice and building regulations. In attempting to prepare codes to reduce the level of risk from abnormal loadings, recent codes throughout the world have taken one or a combination of the following approaches:

- (a) the "cautionary note" which warns the design engineer of the problems;
- (b) the "alternate path" which requires the engineer to explicitly consider removal of any structural element;
- (c) the "specified abnormal loading" which requires the engineer to design against an arbitrary overpressure load; and
- (d) the "general structural integrity" which attempts to secure adequate integrity through minimum detailing practice by establishing a degree of continuity and ductility within and between structural elements.

For purposes of illustration and comparison, the methods used by various countries are described below.

### 3.1 British

Reaction to Ronan Point was immediate. Six months after the disaster the British Government issued regulations intended to alleviate the problem of progressive collapse.<sup>(156)</sup> These regulations, were adopted in 1970 as the 5th amendment to the 1965 Building Regulations.<sup>(157)</sup> These code provisions, which applied to structures over five stories, required that a building be designed by the engineer so that a failure resulting from the removal of any portion of any one structural member be localized within a story and be limited to three stories. A local failure within a story was defined to be 750 square feet or 15% of the floor area.

problem to the engineer.

For economic considerations and for practicality, this hastily developed solution came under much criticism.<sup>(158-166)</sup> As a result, these regulations received considerable attention from code-writing bodies and were significantly modified. Under present British specifications,<sup>(59)</sup> certain minimum details for continuity and ductility are incorporated into the design, the 5 psi or member removal design are not required.

## C.2 French

The French (CEB) regulations<sup>(54)</sup> were among the first codes to recognize the problems of progressive collapse and abnormal loads in LP buildings. Within the introduction there is an immediate cautionary note which states: "One can hardly overemphasize the absolute necessity of effectively joining the various components of the structure together to obviate any possible tendency for it to behave like a 'house of cards' and of organizing it accordingly." Although the regulations also contain further minimum detailing specifications, they are not considered as thorough as those contained in the British Standard.

The French reaction to the Ronan Point disaster was not as volatile as that of the British. It was argued that the CEB regulations already recognized the problems of continuity and ductility in LP buildings. Furthermore, it was noted that the French regulations for gas installations were extremely restrictive, therefore gas explosions in high-rise blocks were considered less likely.<sup>(167,168)</sup> Their first point had been well taken, however, regarding the latter, elimination of the occurrence of one abnormal loading condition is arbitrary considering the many other abnormal events that can occur.

- . Explosions are only one example of so-called excessive loadings, all of which have to be considered.
- . All kinds of buildings, irrespective of construction type and size (height), should be considered.
- . Stability must be considered on the large scale (the total stability of the buildings), and in the details (the keeping of the separate parts together).
- . It is of greater practical importance to limit the scale of the damage than to try to eliminate damage totally.
- . Excessive loadings can be of different magnitudes, and it is desirable to apply precautions that are balanced from a statistical point of view. Excessive loadings are more important in bigger buildings than in smaller ones.

Swedish Regulation draft<sup>(82)</sup> which has now been adopted<sup>(96)</sup> was based on the above recommendations combined with additional background studies.<sup>(78,80,86,98)</sup> It emphasizes the importance of sufficient strength and ductility in all connections of a framework. The Swedish Standard allows excessive loadings to be taken into account in alternate ways: by localizing the damage; by designing with sufficient strength to resist damage; or by decreasing the risk of occurrence of an excessive damage. The main interest, however, is centered on specifications to limit damage and resist progressive collapse by ensuring that both the member forces and the connection design are adequate in size and strength. The regulations are similar in nature and intent to those of the British Standard.

progressive collapse due to local failure are reduced to a level commensurate with good engineering practice.

Although there is a substantial commentary<sup>(83)</sup> to the code provision, the engineer is provided only a qualitative prescription for solving the problem. The code offers no minimum detailing practice to establish such integrity and thus places this burden on the design engineer.

## 5 United States

In the United States there have been two cases where criteria to deal with progressive collapse were adopted. The first was the Guide Criteria<sup>(169)</sup> used in the evaluation of housing systems for the Operation Breakthrough program. The second<sup>(170)</sup> was prepared by the Federal Housing Administration of HUD for use in preparation of Structural Engineering Bulletins for those industrialized systems being developed and built with FHA mortgage guarantees. Both documents were adaptations of the early British regulations and required either the alternate panel method or the specified abnormal load approach (5 psi). As was the case in Great Britain, these early U.S. regulations immediately came under attack from the profession as being too radical, uneconomical, and inappropriate. As a result, the 1971 HUD document was revised a number of times, with the current revision<sup>(171)</sup> issued in May 1974, requiring minimal diaphragm and wall ties, and also, either a 5 psi overpressure design, or a notional removal design.

Contrary to the approaches used in the above documents, the PCI Committee on Precast Concrete Bearing Wall Buildings, has developed a design manual<sup>(53)</sup> which presents design recommendations emphasizing the interdependence of elements and the need to provide continuity and ductility to establish a single coherent assembly. Much of the minimum suggestions





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Convert from	to	multiply by
<b>Length</b>		
inch (in.)	centimeter (cm.)	2.54
inch (in.)	meter (m.)	0.0254
foot (ft.)	meter (m.)	0.3048
yard (yd.)	meter (m.)	0.9144
<b>Area</b>		
square foot (sq.ft.)	square meter (sq.m.)	0.0929
square inch (sq.in.)	square centimeter (sq.cm.)	6.452
square foot (sq.ft.)	square meter (sq.m.)	0.0009
square yard (sq.yd.)	square meter (sq.m.)	0.0361
<b>Volume</b>		
cubic inch (cu.in.)	cubic centimeter (cu.cm.)	16.39
cubic inch (cu.in.)	cubic meter (cu.m.)	0.0000
cubic foot (cu.ft.)	cubic meter (cu.m.)	0.0283
cubic yard (cu.yd.)	cubic meter (cu.m.)	0.7646
gallon (gal.) U.S. Liquid**	liter	4.546
gallon (gal.) U.S. Liquid**	cubic meter (cu.m.)	0.0045
gallon (gal.) U.S. Liquid**	liter	3.705
gallon (gal.) U.S. Liquid**	cubic meter (cu.m.)	0.0033
<b>Force</b>		
kip	kilogram (kg)	453.6
kip	newton (N)	4,448.
pound (lb.)	kilogram (kg)	0.4536
pound (lb.)	newton (N)	4.448
<b>Pressure or Stress</b>		
kip per square inch (ksi)	kilogram per square centimeter (kg/sq.cm.)	70.31
pound per square foot (psf)	kilogram per square meter (kg/sq.m.)	4.882
pound (force) per square foot (psf)	pascal (Pa.)	47.00
pound per square inch (psi)	kilogram per square centimeter (kg/sq.cm.)	0.0703
pound (force) per square inch (psi)	pascal (Pa.)	6,095.
<b>Mass (Weight)</b>		
pound (lb.) avdp.	kilogram (kg)	0.4536
ton, 2,000 lb.	kilogram (kg)	907.2
grain	kilogram (kg)	0.0000
<b>Mass (Weight) per Length</b>		
kip per linear foot (klf)	kilogram per meter (kg/m.)	0.0014
pound per linear foot (plf)	kilogram per meter (kg/m.)	1.400
<b>Mass per Volume (Density)</b>		
pound per cubic foot (pcf)	kilogram per cubic meter (kg/cu.m.)	1
pound per cubic yard (pcy)	kilogram per cubic meter (kg/cu.m.)	
<b>Temperature</b>		
degree Fahrenheit (deg. F.)	degree Celsius (C)	$t_C = (t_F - 32) \times \frac{5}{9}$
degree Fahrenheit (deg. F.)	degree kelvin (K)	$t_K = (t_F + 459.67) \times \frac{5}{9}$
<b>Energy</b>		
British thermal unit (Btu)	joule (J)	1,056
kilowatt-hour (kwh)	joule (J)	3,600,000
<b>Power</b>		
horsepower (hp) 550 ft.-lb./sec.	watt (W)	
<b>Velocity</b>		
mile per hour (mph)	kilometer per hour	
mile per hour (mph)	meter per second (m/s.)	